

A Landslide Risk Management Approach for the  
Stillwater to Ngakawau Rail Corridor (SNL96 to 126km)  
in the Lower Buller Gorge, New Zealand

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**Kristel Franklin**

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This thesis has examined the 30km long rail corridor through the Lower Buller Gorge, on the Stillwater Ngakawau Line, between SNL96 and 126km, using a landslide risk management approach. The project area is characterised by high annual rainfall ( $>2,000\text{mm}$  per year), and steep topography (slopes typically  $\geq 20^\circ$ ) adjacent to the rail corridor. The track formation generally follows the natural contour near the base of the hillslope through the Lower Buller Gorge, and consequently involves many curves but relatively limited cut slopes into adjacent rock outcrops. The distance between the base of adjacent hillslopes and rail is frequently  $<2\text{m}$  horizontally.

A variety of basement and Tertiary lithologies are present, including granite, breccias, indurated sandstone/mudstone, and limestone. The primary focus of this thesis has been on upslope-sourced landsliding onto the rail corridor, and on two short lengths (20m and 450m) that currently have a 25km/hour speed restriction imposed at Whitecliffs and Te Kuha respectively. Rainfall-induced and earthquake-generated landslide triggering mechanisms were examined in detail.

A landslide inventory has been compiled to determine the characteristics and distribution of identified slope failures over time, and to establish any correlation with topography and geology. Sixty individual landslide events were identified since the line became fully operational in the 1940s, based on desktop reviews, and field inspections for more recent events. To reflect the presence of small magnitude landslide events, a project-specific logarithmic classification of landslides was adopted from  $<10\text{m}^3$  (very small volume) to  $\geq 10,000\text{m}^3$  (very large volume). An absence of a higher proportion of ‘very small’ to ‘small’ landslide volumes ( $<100\text{m}^3$ ) in the inventory reflects incomplete reporting of these comparatively lower magnitude, but higher frequency, events. The establishment of a robust landslide inventory to document future events, in a consistent and readily accessible format, is required for continued monitoring and review of landslide risk management practices in the Lower Buller Gorge.

Combining landslide inventory data and physical characteristics of the project area enabled the development of a qualitative landslide zonation map that assigned ‘high’, ‘high-moderate’, ‘moderate’ and ‘low’ landslide susceptibility classes. The principal area of slope instability above the rail corridor is 22.5km in length between SNL103.5 and 126.0km, associated predominantly with basement lithologies (Tuhua Granite; Hawks Crag Breccia; Greenland Group). The most frequently occurring landslides are shallow, typically less than 3m deep, translational failures triggered in regolith or colluvium materials. Rainfall-induced debris slides and flows are dominant, given the high annual rainfall and associated high frequency of high intensity or long duration rainfall events. Very small to medium landslides ( $<1,000\text{m}^3$ ) have the potential to impact the rail corridor with an average frequency of around one every two years, causing damage to infrastructure or affecting rail operations. Very large landslides ( $\geq 10,000\text{m}^3$ ) can be expected every 10 to 20 years based on a limited

historical record. The narrow rail corridor and absence of sufficient catch areas above or adjacent to the rail causes continual operational challenges due to upslope-sourced landslide debris, and high susceptibility to slope failures, particularly west of SNL103.50km. Development of a rainfall-threshold for proactive inspection of the rail corridor is recommended, including the establishment of a rain gauge network through the Lower Buller Gorge.

Earthquake-generated landslides significantly impacted the rail during the magnitude 7.1 Inangahua earthquake in 1968 and to a much lesser extent during the magnitude 6.1 Westport earthquake in 1991. The rail was not fully constructed through the Lower Buller Gorge at the time of the magnitude 7.8 Buller (Murchison) Earthquake in 1929, which generated widespread landsliding in the Buller and Nelson regions. Earthquake-generated landsliding can be expected through the Lower Buller Gorge from earthquakes of magnitude  $\geq 6$ , and track inspection is recommended in the event of magnitude 5 or greater earthquakes.

Detailed geological characterisation and mapping at Whitecliffs and Te Kuha was conducted, including a LiDAR survey at Whitecliffs that enabled visualisation of the ground surface without the interference of vegetation. The limestone outcrop at Whitecliffs comprises 60-70m high near-vertical cliffs with a well-established talus apron at the base, extending to the rail corridor. Three widely spaced open fractures sets are present at the top of Whitecliffs that propagate into the cliff-face. There has been no detectable movement on selected key fracture sets since monitoring commenced in 1993 and there is no confirmed evidence of large-scale cliff collapse during the 1968 Inangahua earthquake. Whitecliffs is not as susceptible to failure as other slopes inspected in the project area due to structural controls, primarily being the dipping of strata back into the cliff-face and widely spaced joint sets. Establishment of inspection protocols for earthquake events impacting the area, including real-time monitoring of selected fractures at Whitecliffs is recommended.

A 2km-length corridor site model produced for Te Kuha demonstrated 'high' landslide susceptibility is not confined to slopes above the existing 450m speed restriction zone. Removal of the speed restrictions at Whitecliffs and Te Kuha can be considered, as the increased exposure time is not considered sufficient justification given the extent of other susceptible areas to landsliding affecting the Lower Buller Gorge rail corridor.

The principal conclusion from this thesis project is that there is on-going risk to rail operations predominantly from shallow translational landsliding in regolith-colluvium materials. The majority of these will be generated by long-duration or intense rainfall events. Development of threshold-based methods for effective track management is recommended, including the establishment of a rain gauge network through the Lower Buller Gorge, and landslide inventory database. Site-specific engineering measures could be adopted, such as catch benches or avalanche-type shelters, where justified on a cost-benefit basis.

I would like to first acknowledge Richard Justice from KiwiRail for introducing me to Whitecliffs and providing the funding, support and opportunity to work in this impressive region. Gaining insight into the world of rail on the West Coast in conjunction with studying hazard management was a unique and very enjoyable experience. The acknowledgment also extends to personnel at the KiwiRail Westport and Greymouth depots who assisted in providing me access into the Lower Buller Gorge during some difficult times in September 2010 and February 2011 only days after two major Canterbury earthquakes.

Secondly, I want to thank my husband, Miles, for making it possible for me to complete my goal of finishing this thesis. Taking the children out when you knew I had deadlines approaching, and supporting me every step of the way made the whole process achievable.

To my thesis supervisor David Bell I thank you for your guidance with the rock mechanics aspects that I needed to develop for the thesis, in addition to grammatical advice! Thank you also to Rouwen Lehné for your patience and support with the ArcGIS component of the thesis, and assistance during site visits to the coast. Abseiling down Whitecliffs was a definite highlight. Completing the thesis in Nelson posed a few logistical challenges at times, but thanks to the support from the Geological Sciences Department in assisting with laboratory testing, thin section preparation and general advice, working remotely was successful.

Thank you to my father, Robin Turton, for being an exemplary field assistant. Our many traverses up to the top of Whitecliffs, waiting at the 121km mark, walking kilometres laden with rocks, driving to Inangahua (then back to Westport, back to Inangahua, around the back of Whitecliffs, up to Te Kuha and then repeating something similar the next day) was made easy due to your unlimited patience and natural interest in the project. To the rest of my family in Nelson and Christchurch I want to express how grateful I am of your amazing support, particularly in the last three months as the deadline approached.



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# CHAPTER 1: INTRODUCTION

## 1.1 Project background

The Lower Buller Gorge rail corridor forms part of the Stillwater to Ngakawau Line (SNL), located between Inangahua and Westport on the West Coast of New Zealand. The project area is defined on Figure 1.1, which incorporates the 30km length of rail corridor between SNL96km (eastern end) and SNL126km (western end). The rail corridor is aligned adjacent to the northern bank of the Buller River. The track formation generally follows the natural contour near the base of the hillslope through the Lower Buller Gorge, and consequently involves many curves but relatively limited cut slopes into adjacent rock outcrops.

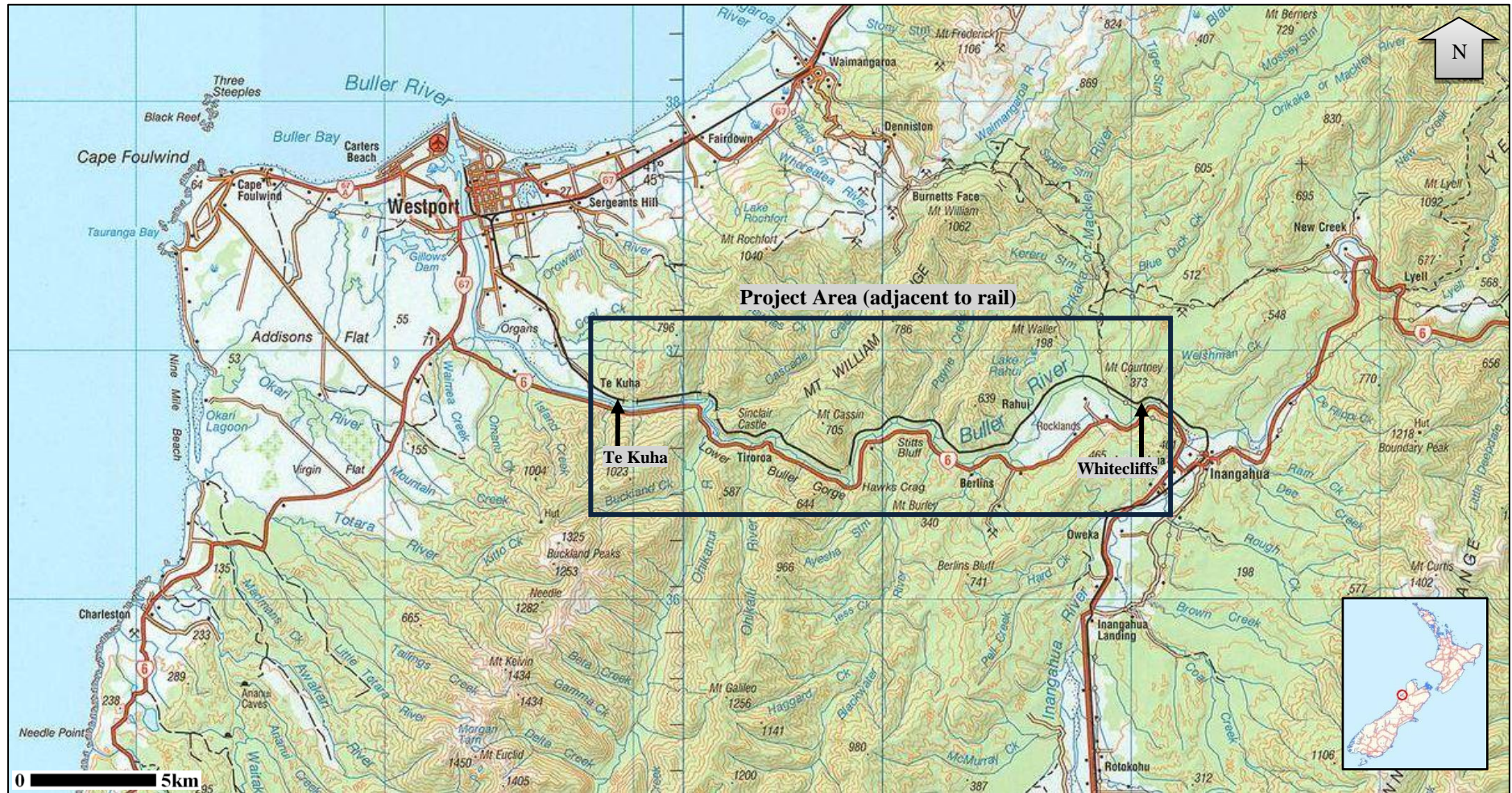
The typically steep terrain, access limitations and climatic conditions (high annual rainfall) in the region pose many challenges to the effective on-going maintenance of the rail infrastructure and operations. Information provided by KiwiRail (ONTRACK Rail Operating Procedures, Section: L6 – page 15) indicate that there are currently two 25km/hour speed restrictions imposed within the project area due to potential slope stability hazards. The general location of the speed restriction areas are indicated on Figure 1.1, labelled as:

- **Whitecliffs:** 20m length of rail between SNL97.510km and 97.530km
- **Te Kuha:** 450m length of rail between SNL124.050km and 124.500km

Whitecliffs and Te Kuha are respectively located near the eastern and western extent of the Lower Buller Gorge rail corridor (Figure 1.1). Speed restrictions within rail corridors have an economic impact on the long-term efficiency of operations. The timing and rationale behind the implementation of the two permanent 25km/hour speed restrictions in the Lower Buller Gorge are not well documented and KiwiRail have expressed an interest in quantifying any geotechnical risk associated with removal of the restrictions. In addition to the two speed restrictions, locomotive engineers are advised in the Rail Operating Procedures to be aware of slope stability hazards between SNL124.256km and 125.500km (1,244m length), with specific reference to rockfalls. This area incorporates a section of the 25km/hour speed restriction zone referred to above at Te Kuha.

To enable a robust assessment of geotechnical risks associated with landslide hazards above the rail alignment, landslide occurrence, characterisation and susceptibility through the entire rail corridor shown on Figure 1.1 has been conducted, including detailed research at Whitecliffs and Te Kuha. This thesis presents the findings of the research and is intended to provide KiwiRail with a tool for use in future Landslide Risk Management in regards to rail operations in the Lower Buller Gorge.



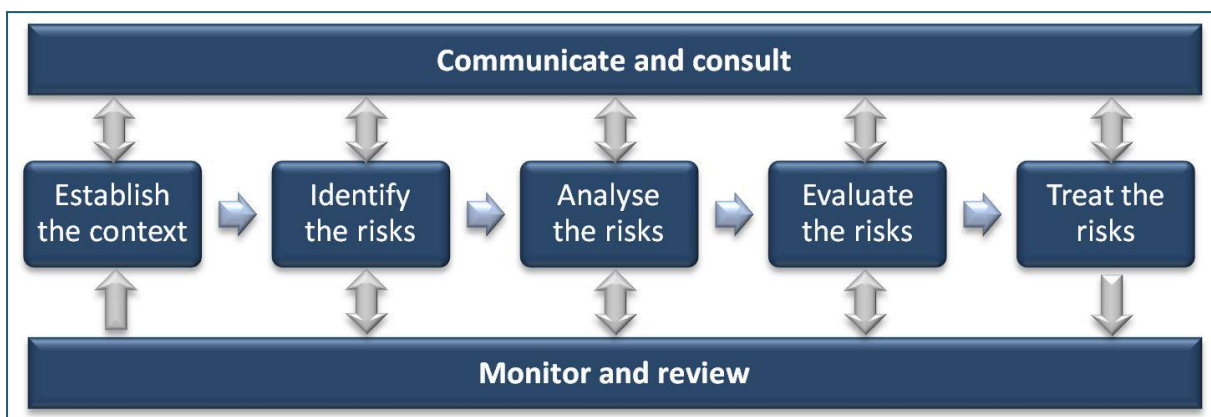


**Figure 1.1: Project area – Stillwater to Ngakawau Line (Lower Buller Gorge) between SNL96km and 126km. Labels shown indicate the location of Te Kuha and Whitecliffs that have a permanent 25km/hour speed restriction. Inset indicates location of the project area (circled) in the South Island, New Zealand.**

## 1.2 Context and methodology

The context for this thesis is landslide risk management. This is based on recognition that landslide hazards in rail corridors pose risks to the safety of locomotive engineers, and other personnel working in these areas. Economic impacts in terms of potential damage to rolling stock, infrastructure and the associated risk reduction measures also need consideration.

The consequences of a landslide occurring can range from the worst case scenario of human fatalities or injuries, to economic effects related to service disruption, recovery and rebuilding. The methodology adopted for conducting the research, and presentation of results in this thesis, follow the general risk management process outlined in Figure 1.2.

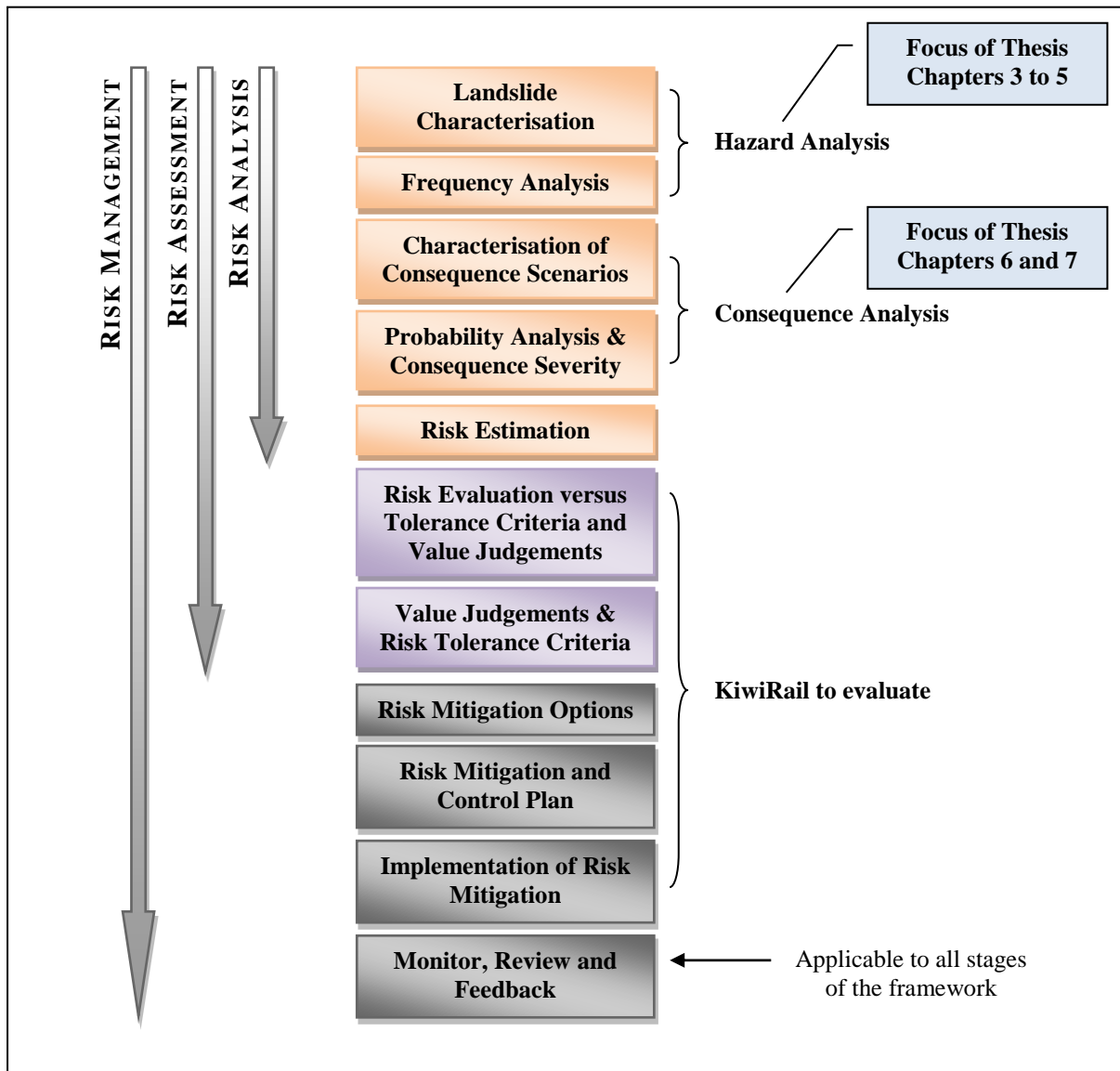


**Figure 1.2: Risk management process (based on AS/NZS ISO 31000, 2009)**

A flowchart produced by the Australian Geomechanics Society (AGS) in 2007 expands the general process outlined in Figure 1.2 specifically for landslide risk management, including detail on the type of methodology and information requirements for each component of the framework. A copy of the framework is provided in Appendix 1.1 for reference, with the main components summarised on Figure 1.3.

The landslide risk management framework was also adopted by the Joint Technical Committee on Landslides and Engineered Slopes (Fell et al, 2008a). This thesis generally follows the guidelines for landslide susceptibility, hazard and risk zoning outlined by Fell et al (2008a), which is weighted strongly towards the work completed by AGS (2007) but also incorporates a uniformity for terminology and quantitative risk management principles. A commentary to the guidelines (Fell et al, 2008b) provided additional background information and references to supplement the original paper. Definitions of terms used by Fell et al (2008a) in regards to landslide zoning and risk management are based predominantly on IUGS (1997). A list of definitions is provided in Appendix 1.2.





**Figure 1.3: Framework for landslide risk management (based on Fell et al, 2008a)**

Risk analysis is the first stage of the Landslide Risk Management framework, which involves hazard and consequence analysis (Figure 1.3). Risk estimation is the outcome of considering hazard and consequence analysis that can be incorporated into a risk assessment, including value judgements and risk tolerance criteria. The scope of the thesis does not include components of the framework subsequent to risk estimation. These aspects, and subsequent stages of the Landslide Risk Management process, require evaluation from KiwiRail, and can be developed further upon completion of this thesis (Figure 1.3).

The occurrence of landslides is coupled closely with a number of possible triggering mechanisms, in the present context primarily being precipitation and seismicity. Hazard analyses consider these mechanisms by interpretation of the data collected and development of a landslide susceptibility map for the project area, including site-specific models for Whitecliffs and Te Kuha.

Recommendations regarding possible mitigation measures and options are summarised where relevant in this thesis. This stage of the risk management process (treating the risks) will be completed, and decisions regarding implementation made, by KiwiRail personnel upon review of findings from this thesis and any other independent studies.

Monitoring and communication is an important component of all risk management processes (Figures 1.2 and 1.3). This has been achieved through progress reports and frequent consultations with KiwiRail, University of Canterbury supervisory staff and liaison with consulting companies involved with other relevant projects in the area.

### 1.3 Research hypotheses

Based on a preliminary assessment of the available information, the original hypotheses for the thesis project were:

- That the cause of rock mass dilation in near horizontally bedded limestone observed in the outcrop at Whitecliffs is at least partly attributable to seismic shaking.
- That the risk of damage to railway infrastructure between SNL96 and 126km is probable, in the event of a large magnitude earthquake, due to rockfalls originating from dilated rock masses on the scale observed at Whitecliffs and other locations.
- That systematic and detailed hazard mapping at Whitecliffs and Te Kuha, and identification of other potentially hazardous locations, will enable an assessment of vulnerable sites and development of appropriate management strategies.

As the thesis project has progressed the original hypotheses have been tested and modifications have resulted to the overall research objectives presented in the following section.

### 1.4 Research objectives

Permanent speed restrictions at Te Kuha and Whitecliffs are in place to protect the safety of personnel working in these areas, and assist in minimising any disruption to rail operations. The effectiveness, reliability and actual need for the speed restrictions have not been quantified to date. Based on the limited information currently available on rockfall hazards, and other slope instability issues affecting the rail corridor through the Lower Buller Gorge, the research objectives adopted for this thesis are:

- **Objective A:** Systematic identification and documentation of landslide hazards within the project area in order to provide an overall context for hazard management in the Lower Buller Gorge.



- **Objective B:** To identify, and quantify, geological and geotechnical risks at Whitecliffs and Te Kuha, with specific reference to the removal of 25km/hour speed restrictions.

Based on the information obtained, risk management strategies and recommendations for mitigation options have been developed. Identification of areas that require further work is also highlighted.

## 1.5 Thesis structure

The thesis is structured into the chapters summarised in Table 1.1 to provide a coherent presentation of the research conducted. Each chapter includes an introductory section and a summary of main findings and/or limitations encountered.

**Table 1.1: Thesis chapter summary**

Chapter	Title	Purpose(s)
1	Introduction	Detail presented on project background; thesis scope and methodology; Lower Buller Gorge project area, and background information (current operations, rail terminology and infrastructure); fundamentals of landslide hazards; background geotechnical report. Thesis objectives and research hypotheses based on preliminary information available.
2	Physical Setting	Outline of the project area, including climatic conditions; geomorphic characteristics (tectonic processes, topography and watercourses); geology and seismicity.
3	Landslide Inventory	A) Determine bounding conditions in regards to the timing, and rationale, for the speed restrictions at Whitecliffs and Te Kuha. Achieved through literature reviews, and interviews with current and former rail personnel. B) Documentation of all available data sources on landslide hazards and occurrence within the rail corridor, including historic aerial photograph review, KiwiRail data, West Coast Regional Council (WCRC) datasets and other published records;
4	Landslide Susceptibility	Presentation of the spatial distribution of landslide occurrences. Correlation of landslide occurrence with topography and geological controls. Presentation of a rail corridor landslide susceptibility map using four relative categories for zonation (Low to High Susceptibility).
5	Landslide Hazard	Landslide characterisation and frequency analysis, including presentation of representative case studies involving large volume ( $>1,000\text{m}^3$ ) events. Detailed discussion on rainfall-induced and earthquake-generated landslides. Temporal case study related to impacts from a long duration rainfall event in December 2010.

Chapter	Title	Purpose(s)
6	Whitecliffs	Geological mapping, site investigations, laboratory testing and collation of all data to develop a site model for Whitecliffs in regards to slope instability. Assessment of the 25km/hour speed restriction.
7	Te Kuha	Geological mapping, site investigations, laboratory testing and collation of all data to develop a site model for Te Kuha in regards to slope instability. Consequence analysis conducted relevant to the 25km/hour speed restriction.
8	Future Rail Management	Outline best practice management options and protocols related to landslide risk management in the Lower Buller Gorge rail corridor between SNL96 and 126km.
9	Summary and Conclusions	Presentation of a concise summary of the main thesis findings, conclusions and future work recommendations.

## 1.6 Project area

The 30km length of rail corridor that forms the project area shown on Figure 1.1 comprises a dynamic physical environment that poses many challenges to the on-going effective operation of commercial rail services. This section provides an outline of the current rail operations, infrastructure and rail-related terminology referred to frequently in this thesis. Whitecliffs and Te Kuha are introduced in this section to provide background information on the two locations currently with 25km/hour speed restrictions imposed.

### 1.6.1 Current operations

There is currently an average of six train movements each day through the Lower Buller Gorge. The majority of trains carry coal sourced from the Stockton Opencast Mine, located approximately 35km north of Westport, and operated by Stockton Alliance (a partnership between Solid Energy and Downer EDI Mining New Zealand). A small volume of cement from the Holcim New Zealand Westport Plant is also transported through the gorge on a regular basis but the majority of cement from Holcim's operations is carted by cement tankers from their marine terminal. There are currently no passenger trains operating within the Lower Buller Gorge and it is unlikely that any regular tourist or commuter operations will be established in the foreseeable future.

A team of rail personnel, based in Westport, work in the Lower Buller Gorge rail corridor using high-rail inspection vehicles for general maintenance and track upgrading work. In addition, KiwiRail-approved contractors also work in the rail corridor for specific projects, including vegetation

clearance and civil works, as required. Track inspections (wet weather runs) are conducted after high intensity and/or long duration rainfall events to ensure the rail is safe for personnel and rolling stock. Structural inspections also occur in the event of an earthquake. It is understood that KiwiRail do not currently have established thresholds for determining the requirement for these inspections in response to triggering mechanisms that have the potential to cause slope failures or other track integrity issues.

An independent project, separate from this thesis, is in progress to characterise slope stability issues through the gorge and development of rainfall thresholds to formalise the implementation of track inspections. It is understood that there is no inclusion within the independent project for specifying seismic triggers at this stage.

### 1.6.2 Metrage pegs

To determine locations within the project area the most common reference points used in this thesis are metrage pegs. In the rail corridor, these are triangular marker pegs labelled with a distance reference every kilometre. The SNL125km mark is shown in Figure 1.4.



**Figure 1.4: Metrage peg example (SNL125km)**

In the Lower Buller Gorge the number on the metrage peg refers to the kilometre distance from Stillwater, located approximately 15 kilometres east of Greymouth. The 500m point between two metrage pegs is indicated by a yellow triangular peg. Bridges, tunnels, signs and signals may also display metrage references to three decimal places (metres). Metrage references are specific to the rail line in question, in this case the SNL.

### 1.6.3 Track warrant control zones

There are seven track warrant control (TWC) zones that cover the SNL within the Lower Buller Gorge project area. These zones define working areas along the rail alignment. When communicating a location to train control, located in Wellington, reference is made to the line (SNL), TWC zone, and metrage pegs.

Table 1.2 provides a reference for distinguishing between the TWC zones in the Lower Buller Gorge. The seven TWC's are also shown on Figure 1.5. As indicated by the 'length' column in Table 1.2, the zones are typically between 3km and 6km. It is important to note that the area referred to as Te Kuha in this thesis is actually within the Cascade TWC. The Te Kuha TWC starts from SNL126.50km, outside of the project area. Whitecliffs is located within the Buller TWC.

**Table 1.2: Track warrant control areas (SNL96.0 to 126.5km)**

TWC area	SNL metrage (start of TWC)	Length (km)
Buller	93.50km	6.00
Mackley	99.50km	3.70
Rahui	103.20km	2.70
Berlins	105.90km	4.25
Twin Bridges	110.15km	5.85
Tiroroa	116.00km	4.50
Cascade	120.50km	6.00 (to Te Kuha)

Reference to rail localities within this thesis arise from a variety of sources, including geological units (e.g. Hawks Crag Breccia); topographical features (e.g. Sinclair Castle) or, in the absence of a specific feature, the TWC zone is used. This is particularly relevant for the Berlins, Rahui and Mackley TWCs (Figure 1.5) where other topographical or geological references do not dominate.



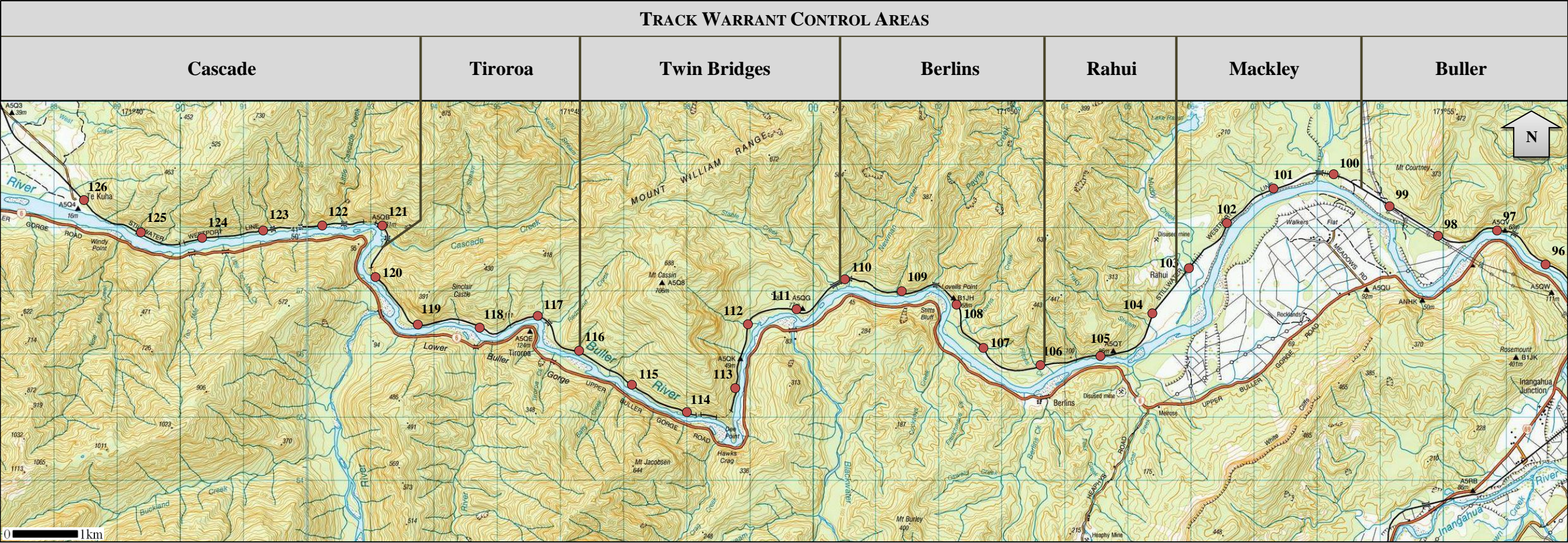


Figure 1.5: Project area between SNL96 and 126km, showing metrage references and track warrant control zones  
Source: New Zealand Topographic Maps (NZTM Sheets B20 and B21), 1:50,000



### 1.6.4 Infrastructure

Detail regarding tunnels and bridges within the project area was obtained from a geographical information system (GIS) provided by KiwiRail (valid as of 9 September 2010). A summary of the main details from the GIS, and other infrastructure information sourced from a publication by the New Zealand Railway and Locomotive Society Incorporated (NZRLSI, 1964), is provided below.

- **Bridges:** The GIS lists 21 bridges within the project area, which includes structures over natural streams, gullies and culverts. Construction materials vary. Details for some of the comparatively larger structures, respective rivers that they cross, and eastern SNL start metrage, are:
  - 100.289km, Bridge 92 (Orikaka/Mackley River): Steel bridge.
  - 108.460km, Bridge 97 (Slaty Creek): Concrete arch bridge.
  - 109.923km, Bridge 98 (Newman Creek): Concrete arch bridge.
  - 110.129km, Bridge 99 (Stable Creek): Concrete arch bridge.
  - 116.779km, Bridge 100 (Redmond Creek): Concrete arch bridge.
  - 120.746km, Bridge 102 (Cascade Creek): Steel girder structure on high piers.
- **Tunnels:** Five tunnels are present between SNL113km and 125km. The longest tunnel, located near SNL113km (Tunnel 2), is formed through Hawks Crag Breccia and is 260m in length. Tunnel numbers, start and end metrage references on the SNL, and corresponding lengths in brackets, for existing tunnels are summarised below:
  - Tunnel 2: 113.338 and 113.598km (260m)
  - Tunnel 3: 113.855 and 113.909km (54m)
  - Tunnel 4: 120.100 and 120.252km (152m)
  - Tunnel 5: 121.178 and 121.329km (151m)
  - Tunnel 6: 124.208 and 124.256km (48m)

Tunnel 7 was originally located near SNL125.40km. This tunnel was daylighted in 1987 after a period of rainfall that resulted in major slope movement above the rail alignment.
- **Retaining structures:** There are no detailed records of retaining walls available but there are numerous structures within the project area, of varying ages and construction materials, for the purpose of retaining ballast, fill material or mitigating slope stability issues above the rail.
- **Grades:** The steepest track gradient was initially 1 in 32 just east of Te Kuha (NZRLSI, 1964). This was subsequently reduced to 1 in 120 and remains the steepest grade heading east (with the exception of a short 1 in 90 section prior to Inangahua).

- **Curvature:** Between the Te Kuha TWC and Rahui TWC there are approximately 100 curves in the rail alignment, with only 30 curves for the next 21km (NZRLSI, 1964), which extends eastwards away from the project area. The average curve radius is reported around 200m, and the overall range between approximately 150m and 800m.

There are currently no early warning systems established within the project area relating to landslide hazards. Rain gauges and water level monitoring data for the region are available via a network operated by the WCRC, but there are no gauges within the project area itself. Information regarding the WCRC network is detailed in Chapter 2, Section 2.2.

### 1.6.5 Whitecliffs

The risk of rockfalls from Whitecliffs is immediately apparent due to the presence of near vertical limestone cliffs, detached blocks and a talus apron located adjacent to the railway (Figure 1.6). Metal pins are located across selected displaced limestone blocks at the top of Whitecliffs for the purpose of monitoring any movement over time. Displacement monitoring has generally been undertaken by KiwiRail personnel annually since December 2001, with the first monitoring round conducted in January 1993.



**Figure 1.6: View of Whitecliffs. Photograph taken from State Highway 6, looking north across the Buller River.**

There has been no movement measured between the selected limestone blocks at the top of Whitecliffs to date. A summary of all monitoring results, and the methodology adopted by KiwiRail, is provided in Appendix 1.3, including photographs of the metal pin locations. The frequency of monitoring, and the absence of telemetered information, will not enable an early warning of a large-scale cliff collapse, but has been useful to show that the seemingly precarious and detached blocks at Whitecliffs have been stable under current environmental and climatic conditions for at least the past 18 years.

### 1.6.6 Te Kuha

The scale of the potential landslide hazards at Te Kuha is not as immediately evident (Figures 1.7 and 1.8). Slope failures are known to have impacted rail operations within the speed restriction area between SNL124.05km and 124.50km. Tunnel 6 is located within the 450m zone and large slope failures ( $\geq 10,000\text{m}^3$ ) are evident on the western side of this tunnel. In addition, there have been minor cuts to form the rail alignment around Te Kuha and it is this human element interacting with the physical environment that has exacerbated the risk of failure by daylighting discontinuities in cut slopes. Detailed mapping and landslide characterisation at Te Kuha is presented in Chapter 7.



**Figure 1.7: View west from SNL124.0km (start of the speed restriction area).**





**Figure 1.8: View east near SNL124.5km (end of speed restriction area).**

## **1.7 Landslide hazards**

This section introduces the fundamental basics of landslide hazards in regards to causes, processes and common terminology. On an international scale landslide hazards have historically caused significant loss of life, damage to property, infrastructure and economic costs. Economic losses can be either direct costs associated with repair and maintenance work; or indirect costs, including implementation of mitigation measures to reduce impacts from future landslide occurrences and/or loss of productivity. A trend for increasing susceptibility for landslide hazards over time is considered by Schuster (1996) to be as a result of:

- Increased urbanisation and development in marginal environments that are prone to landslides;
- Deforestation, which increases landslide frequency; and
- Increased regional precipitation due to changing climate conditions.

Within the project area it is only the third aspect that has the potential to increase landslide occurrence rates over time as there is limited vegetation removal in the Lower Buller Gorge, and further development adjacent to the rail corridor is not feasible for urban land use or other development.

### 1.7.1 Landslide causes and processes

In terms of understanding the causes for slope movements, and the associated hazards, it is important to recognise that the processes governing slope stability are complex. Both ambient and triggering factors determine slope stability. Within the project area the ambient factors comprise the existing geological structure, geomorphic landform expression and climatic conditions. A hazard only exists due to the presence and current use of the rail corridor. Human activities associated with track formation can potentially exacerbate pre-existing physical characteristics within the project area, including daylighting joints, fractures and/or bedding planes in rock cuts; destabilising old landslide features during track formation and mitigation measures required to control overland flow of water.

Cruden and Varnes (1996) infer that of the many causes influencing slope movement potential, there is only one triggering factor, being an external stimulus. Rainfall-induced and earthquake-generated triggering mechanisms for slope movement are the most dominant processes in the project area with the potential to create hazards for rail users and operational procedures. An inventory of reported landslides within the 30km length of rail corridor, and associated triggering mechanism, is documented in Chapter 3.

The transfer of geological material in a predominantly downslope direction under the direct influence of gravity is the underlying principle for slope movement (Cruden and Varnes, 1996). Landslide processes recognised by Varnes (1978) are broadly characterised by those that have the potential to result in:

- (1) An increase in shear stress;
- (2) A contribution to low strength of materials; or
- (3) A reduction in the material strength.

Earthquakes and tectonic activity can increase shear stress in a region. Low strength of materials is influenced by discontinuities and other rock mass defects that develop over time. Weathering is the main process associated with reducing strength of slope materials.

### 1.7.2 Landslide terminology and types

A definition of the term landslide provided by Cruden (1991) is: *'the movement of a mass of rock, debris or earth down a slope'*. Following from this definition, slope movement processes involve the failure of material, either of mantling soils or rock, by one or a combination of the following styles (based on Varnes, 1978):

- **Falls:** free fall through the air, with possible leaping, bounding or rolling of fragments.
- **Topples:** pivotal rotation of material around the centre of gravity.
- **Slides:** shear displacement along one or several discrete surfaces, or within a relatively narrow zone. Slide movements are typically rotational or translational.
- **Lateral spreads:** lateral extensional movements either without a basal shear surface or due to liquefaction.
- **Flows:** extremely slow and non-accelerating movement in bedrock; slow to rapid viscous movements in soils.

Consideration of the type of material dictates the full terminology, or forming name, adopted. When the original material is rock, the above styles of movement are simply classified as rockfalls; rock topples; rock slump (rotational slide); rock block or rock slide (both defect-controlled); rock spread and rock flow. The term ‘debris’ is used by Varnes (1978) for predominantly coarse soils that contain greater or equal to 20% of individual fragment sizes larger than 2mm. For soils containing greater than 80% of fine material (less than or equal to 2mm in size) the term ‘earth’ is used.

Distinction between the rates of movement is also commonly used to define slope movements from extremely slow (less than 0.06m per year) to extremely rapid (greater than 3.0m per second). Descriptions for landslides should incorporate the initial movement type and the style of any subsequent movements. A glossary for forming names of landslides from Cruden and Varnes (1996), including the terminology for slope movements listed above, is provided in Appendix 1.4. The term landslip is frequently used in New Zealand but the term is difficult to define and not unanimously supported. The absence of quantifying triggering mechanisms in rock mass descriptions is considered erroneous by Pantelidis (2009). An example of a new system for quantifying slope hazards associated with rock cuttings based on rating tables from Pantelidis (2009) and subsequent work by Pantelidis (2010) is provided in Appendix 1.5.

## 1.8 Pacific Geotech Limited (PGL) Report

Information provided by KiwiRail when the thesis was initially set up included a geotechnical assessment of the SNL between 90km and 126km (PGL, 2007). This section of the SNL was targeted due to it being considered the most at risk from geotechnical hazards. The assessment included identification of the main geological hazards within the rail corridor, and broadly classified these into the following slope instability types:

- **Large-scale cliff collapse:** This mechanism of failure was specifically related to the near vertical limestone outcrop immediately adjacent to the rail alignment at Whitecliffs. The report noted that Whitecliffs has been recognised as an at risk area since the 1950s, but does not provide an information source to justify the timing.
- **Rockfall:** Loose discrete blocks that have the potential to impact the rail alignment. The scale and frequency of this type of failure was considered highly variable.
- **Rainfall-induced rock debris flows:** This type of failure is regarded by PGL (2007) as particularly common in the project area when rock sources mobilise within steeply graded streams above the rail during high intensity rainfall events. The volume of debris mobilised is controlled by the catchment area, rainfall duration and intensity.
- **Instability in weathered surficial materials:** Failures of this nature according to the authors of the report typically occur due to oversaturation of material located on over-steepened and low strength cut slopes.
- **Structurally-controlled landsliding:** Associated with the geological structure of cut slopes and batters. The report identified that this type of failure is of most concern where bedding planes have daylighted in cut slopes, particularly in low strength sedimentary rocks.

Issues associated with drainage, including culverts, embankments and box cuttings, were also examined in the PGL (2007) report. Inadequate (under-sized) culverts are unable to effectively manage overland water flow. Surface flooding of the rail at a number of localities does occur during long duration rainfall events. The importance of adequate drainage design is recognised, including the recommended upgrades at localities identified by PGL (2007), but these issues are not specifically included within the scope of the current thesis.

### 1.8.1 Key risk sites

PGL (2007) concluded that there is a high risk of rockfall or block failure from Whitecliffs at SNL97.520km that will progressively increase over time as rock mass strength is reduced through continued weathering and seismic activity. Stabilisation of the cliff-face and rockfall barriers were not considered feasible mitigation options due to the height of the cliffs (Figure 1.3) and potentially large volume of individual rocks released ( $>200\text{m}^3$ ). The development of an updated monitoring regime was recommended as the most appropriate measure to address this perceived risk, including a remote alarm monitoring system. The current monitoring regime and results to date are presented in Appendix 1.3.

A summary of additional key risk sites identified by PGL (2007) are outlined below from east to west through the project area. Locations are referred to using the relevant SNL metrage and a brief description is provided.

- 98.450km: Failing retaining wall.
- 101.901km: Eroding culvert inlet and outlet.

- 106.650km: Landslide risk, stabilisation required.
- 107.550km: Landslide risk, stabilisation required.
- 114.221km: Embankment determined to have a medium to high risk of failure due to inadequate drainage.
- 118.500km: Rainfall-induced rock debris flows.
- 118.900km: Inadequate drainage through narrow box cutting.
- 122.200km: Inadequate drainage through narrow box cutting.
- 123.850-124.050km: Inadequate drainage through narrow box cutting.
- 124.300km: Rainfall-induced rock debris flows.
- 124.500km: Inadequate drainage through narrow box cutting.

The speed restriction at Te Kuha incorporates the site identified by PGL (2007) at SNL124.300km. The permanent speed restriction in this area is not mentioned. An entry in the summary table appended to the report also includes SNL125.400km as a site that has experienced a large historic landslide. This feature at SNL125.400km (daylighted Windy Point Tunnel) is classified by PGL (2007) as a medium risk site with a recommendation of monitoring the landslide.

Detail in the PGL (2007) report provided an introduction to the location and types of slope instability issues that had been recognised within the rail corridor to date. The PGL (2007) report focussed on rainfall as the predominant triggering mechanism for slope failures between SNL90 and 126km. Seismic activity was not considered, aside from acknowledging the risk of failure at Whitecliffs during an earthquake.

## 1.9 Summary

KiwiRail have expressed an interest in research through the Lower Buller Gorge rail corridor between SNL96 and 126km for the purpose of characterising two areas that currently have a 25km/hour speed restriction imposed (Whitecliffs and Te Kuha). To enable a robust assessment of the entire project area, the Landslide Risk Management framework adopted by AGS (2007) and Fell et al (2008a) has been reviewed and the general approach adopted for this thesis.

A review of landslide hazards, including theoretical information, and a site-specific geotechnical assessment completed by PGL (2007), has enabled key research hypotheses and objectives to be set. Rainfall-induced and earthquake-generated landslides appear the dominant triggering mechanisms for slope movement in the project area. To determine the susceptibility to, and hazards associated with, landslides in the Lower Buller Gorge, the physical environment needs to be understood and previous landslide occurrences documented.

## CHAPTER 2: PHYSICAL SETTING

### 2.1 Introduction

The physical environment considered in Chapter 2 includes climate, geomorphology and geology. Interactions between these elements produce the landscape, vegetation and natural processes that characterise, and continue to shape, the Lower Buller Gorge. An outline and discussion of these physical characteristics, and associated processes, is presented in the following sections to provide an understanding of the context for Landslide Risk Management within the project area.

### 2.2 Climate

Information from the National Institute of Water and Atmospheric Research Ltd (NIWA) indicates the average annual rainfall for most areas in New Zealand is between 600mm and 1,600mm. In comparison, the West Coast region experiences comparatively higher annual rainfall of between 2,000mm and 4,000mm. The mean annual rainfall reported for Westport by NIWA in the period 1971 to 2000 is 2,274mm. Ambient air temperatures are generally mild, with a range for Westport typically between 5°C in winter and 20°C in summer.

Information regarding mean annual rainfall and river water levels specific to the project area was obtained from the WCRC website<sup>1</sup>. The WCRC has a series of water level and rainfall recorders in the region that collectively form a flood warning network. There are three recorders within, or in close proximity to, the project area. Table 2.1 provides a summary of the information available from the WCRC website relevant to the project area.

High flood water levels of 6.7m (Landing) and 8.4m (Woolfs) were recorded in May 1988. Typical water levels for these two sites are 1.0m and 1.2m respectively (Table 2.1). In August 1970 a high flood level of 11.8m was recorded at Te Kuha, which has a typical level of around 1.4m. Water level rises at a rate of 0.4m per hour are reported by WCRC as not uncommon at the Te Kuha recorder site during periods of high intensity rainfall in the southwest-northeast trending Paparoa Ranges. This data highlights the importance of recognising hazards associated with rainfall-induced slope stability issues within the West Coast region.

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<sup>1</sup> WCRC Website: [http://www.wcrc.govt.nz/river\\_level\\_rainfall/24\\_hour/river\\_level\\_rainfall\\_site\\_map.htm](http://www.wcrc.govt.nz/river_level_rainfall/24_hour/river_level_rainfall_site_map.htm). Accessed in February 2011.

**Table 2.1: West Coast Regional Council water level and/or rainfall recorder information**

Recorder name	Location and description	Catchment area	Water levels (summer low and typical levels)	Mean annual rainfall
Landing	<b>Inangahua River</b> , 10km upstream of the Inangahua and Buller River confluence	~1,000km <sup>2</sup>	0.7m 1.0m	2,460mm (daily falls up to 170mm)
Woolfs	<b>Buller River</b> , 3km upstream of the Inangahua and Buller River confluence	~4,560km <sup>2</sup>	0.8m 1.2m	Not recorded (no rainfall gauge at the site)
Te Kuha	<b>Buller River</b> , 16km upstream of the Westport township	~6,350km <sup>2</sup>	0.9m 1.4m	

## 2.3 Geomorphology

Geomorphology considers the surface expression of landforms and the associated earth surface processes that have shaped them. The predominant landforming processes that have shaped, and continue to characterise, the Lower Buller Gorge are tectonic, fluvial and hillslope processes. The tectonic setting is outlined in this section, including a brief outline of topography and steep catchments within the project area. It is also recognised that mass movement processes, including sediment transportation and deposition, are constantly active in any landscape and that these determine the comparatively more recent surface expression of landforms, and influence the geotechnical properties affecting slope stability.

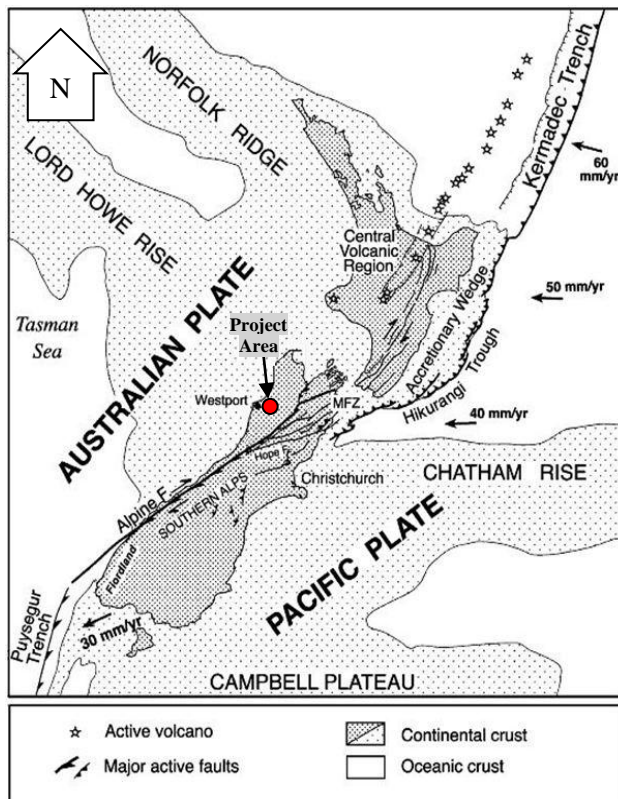
### 2.3.1 Tectonic setting

The tectonic setting of the project area in the context of New Zealand is shown in Figures 2.1 and 2.2. The South Island straddles the Australian and Pacific Plates (Figure 2.1). The collisional movement between the plates is oblique in the South Island. The right lateral movement between the two plates is predominantly accommodated by the Alpine Fault, which has average slip rates of 25-30mm per year (Ghisetti and Sibson, 2006). The geomorphic expression in landforms due to the continued plate motion in the South Island is mountainous terrain, including the Southern Alps that are bounded to the west by the Alpine Fault.

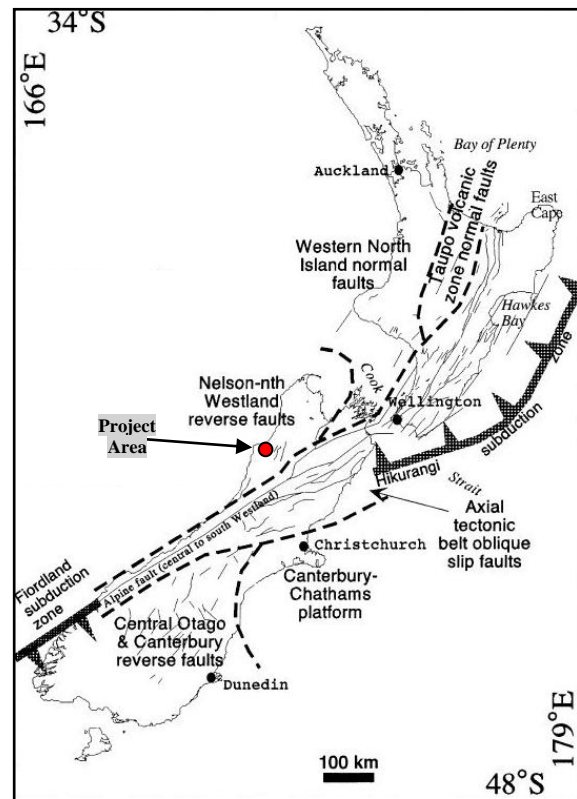
The Lower Buller Gorge is located on the western side of the Alpine Fault in a tectonically active region. Tectonic provinces shown on Figure 2.2 indicate the project area is within the Nelson-North Westland Province, characterised by reverse faults. Tectonic processes on a local scale are directly



linked to the underlying geological structure and lithologies, including igneous intrusions and eruptive sequences. Lithological units present within the project area are outlined in Section 2.5.



**Figure 2.1: Plate tectonic setting of New Zealand (based on Stafford et al, 2008)**



**Figure 2.2: Tectonic provinces in New Zealand (based on Stirling et al, 2002)**

Research conducted by Ghisetti and Sibson (2006) looked at the crustal architecture in the northwest region of the South Island. The research was based around reconstructing structural contours at the base of the Oligocene carbonate sequence to determine how compressional inversion is accommodated in this region. The findings of the research are relevant to this project in terms of understanding the tectonic processes that have formed the current landscape in the region and the implications for seismic activity. A summary of the tectonic history and interpretations made by Ghisetti and Sibson (2006) are provided below:

- **Late Cretaceous-Paleocene and Eocene:** period of extension in pre-existing basement, and the development of high-angle faults (dipping at  $>60^\circ$ ).
- **Early Miocene:** compressional inversion of the high-angle faults due to right-lateral displacement and transpression on the Alpine Fault. Faults typically trend between N-S and NNE-SSW, and dip at the surface between  $45^\circ$  and  $75^\circ$  both to the east and the west. The active reverse faulting cuts sub-parallel folds that deform Tertiary sequences overlying Paleozoic-Mesozoic basement rock.



The current tectonic processes occurring in the region are considered by Ghisetti and Sibson (2006) to involve a mixed style of inversion. The style inferred comprises reactivation of a number of high-angle normal faults and thrusting on comparatively more recent, moderately-dipping and cross-cutting faults. The thrusting results in segments of basement rocks becoming detached, as well as flexural folding of overlying sedimentary rocks. Seismic activity within basement rocks at shallow depths (10-15km) was determined as likely controlled by these potentially blind or concealed faults.

Understanding the tectonic regime in the northwest of the South Island provides a context for the historic seismic activity experienced in the region and the probability of future earthquake events. In terms of a slope movement triggering mechanism, seismic activity is a comparatively low frequency but potentially high impact hazard. Seismic data relevant to the project area from 1900 to 2010 is presented in Section 2.5.

### 2.3.2 Topography and rivers

The topography of the hills above the rail is typically very steep, bush-covered slopes, with the dominant vegetation type of beech and dense podocarp forests. Due to the combination of steep topography and high annual rainfall, the project area is potentially subject to each mechanism of failure outlined in Chapter 1, Section 1.7 (falls, topples, slides, lateral spreads and flows). Falls and slides appear to be the dominant mechanisms of failure in the project area, based on the review of PGL (2007).

The rail alignment follows the natural topographic contours near the hillslope base in the Lower Buller Gorge, above the true right bank of the Buller River. New Zealand Topographic Maps (NZTM Sheets B20 and B21) at a scale of 1:50,000 (Figure 1.5) show the rail alignment within the project area is at an elevation between 50m amsl (eastern end) and 15m amsl (western end). Figure 1.5 indicates 34 watercourses that transect the rail. The majority of watercourses shown are unnamed tributaries to the Buller River. Those that are named are detailed below, moving from east to west through the project area, including the closest SNL metrage peg for reference:

- Welshman Creek (97km)
- Mackley River (100km)
- Muddy Creek (103km)
- Tracy Stream (104km)
- Filtons Creek (106km)
- Browns Creek (108km)
- Payne Creek (109km)
- Newman Creek (110km)

- Stable Creek (110km)
- Redmond Creek (117km)
- Cascade Creek (121km)
- Little Cascade Creek (122km)

Due to the steep catchments in the project area, the watercourses have the ability to erode and transport often large volumes of rock and finer sediment. The deposition of the material forms alluvial fans on the Buller River floodplain. The largest alluvial fan in the project area is associated with deposition from the Mackley River, near SNL100km. In terms of landslide hazards the development of alluvial fans is not in itself a concern, but fluvial processes associated with overland flow of water during high intensity rainfall can result in rainfall-induced slope movements. Saturation of soil also causes shallow regolith failures in cut or steep natural slopes adjacent to the rail.

## 2.4 Geology

The geological structure of units, degree of weathering and impact of cut slopes to form the rail alignment is critical to characterising hazards associated with slope movement within the project area. This section introduces the main geological units present and known fault traces in close proximity to the rail alignment. Seismic data relevant to the project is presented in Section 2.5.

Previous research related to geological characteristics in the Lower Buller Gorge include: Wellman (1950); Beck et al (1958); Nathan (1978); Nathan et al (1986); Tulloch and Kimbrough (1989); and Tulloch and Palmer (1990). Hawks Crag Breccia is one of the most extensively researched units in the Lower Buller Gorge, largely due to the presence of uranium and the associated prospecting that occurred from the 1950s (Beck et al, 1958; Wodzicki, 1959). The following excerpt is from Beck et al (1958) and relates directly to observations in rail cut slopes in the vicinity of Sinclair Castle (shown on Figure 2.6):

*“In the railway cutting near Sinclair’s Castle a radioactive quartz veinlet containing uraniferous hydrocarbon cuts granite-gneiss, on the surface of which meta-autunite is not uncommon. Radioactivity has also been detected in quartz-pyrite veins in granite-gneiss in Mispickel Creek, near Sinclair’s Castle, and in quartz-pyrite-molybdenite lodes in Greenland rocks in Quartz Creek, a tributary of Cascade Creek”*

Mispickel and Quartz Creeks are not major tributaries and their exact locations are unknown but Cascade Creek is shown located between SNL120 and 121km. Beck et al (1958) also refer to at least ten uraniferous horizons on the northern side of the Buller Gorge that range between approximately 50mm and 600mm in width.

Published geological maps that incorporate the project area include Nathan et al (2002); Rattenbury et al (1998); Nathan (1978) and Bowen (1964). Figures 2.3 and 2.4 show the mapped geological units present in the Lower Buller Gorge, based on Bowen (1964) and Nathan et al (2002) respectively. Metrage references are labelled every 5km, denoted also by a blue symbol.

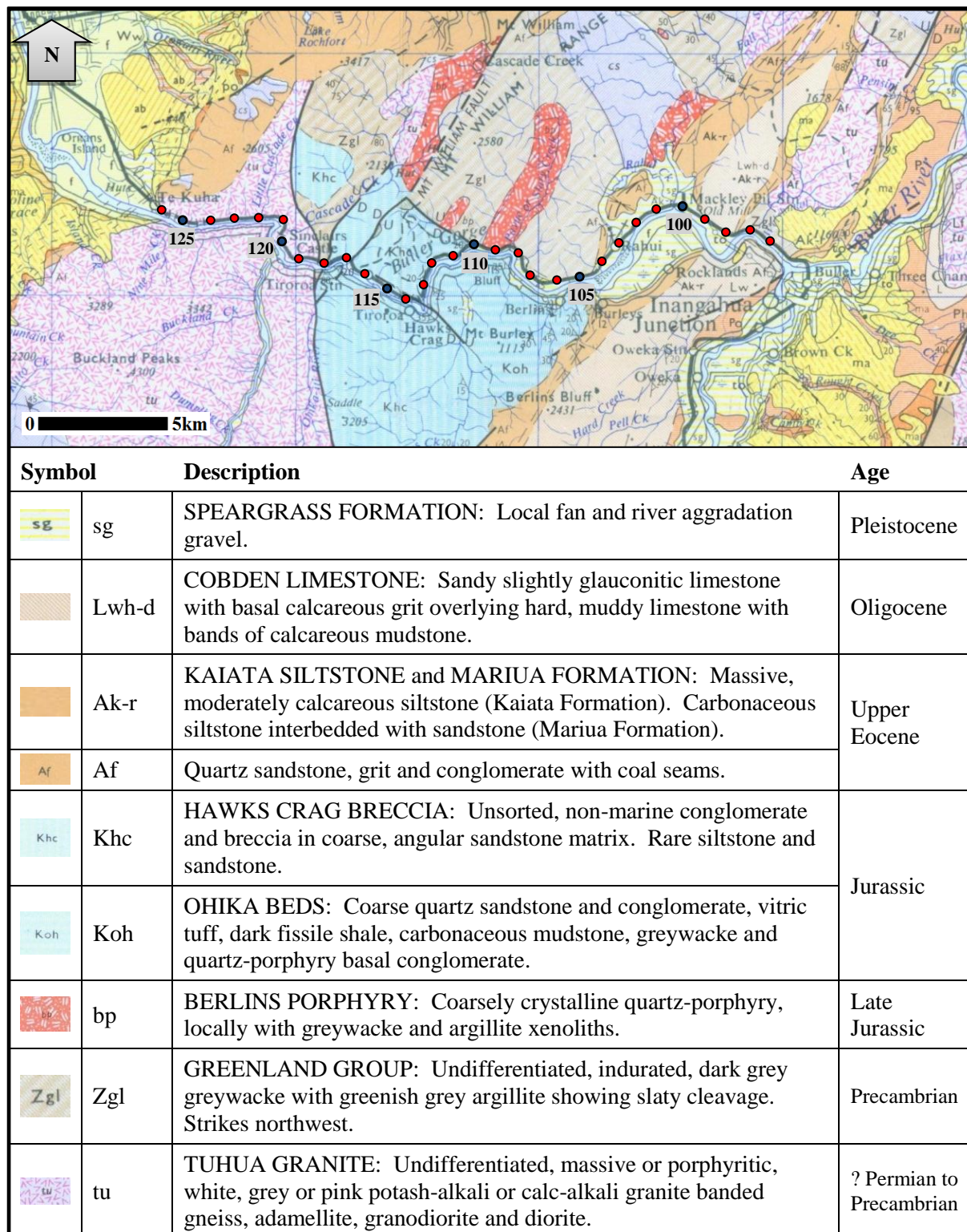
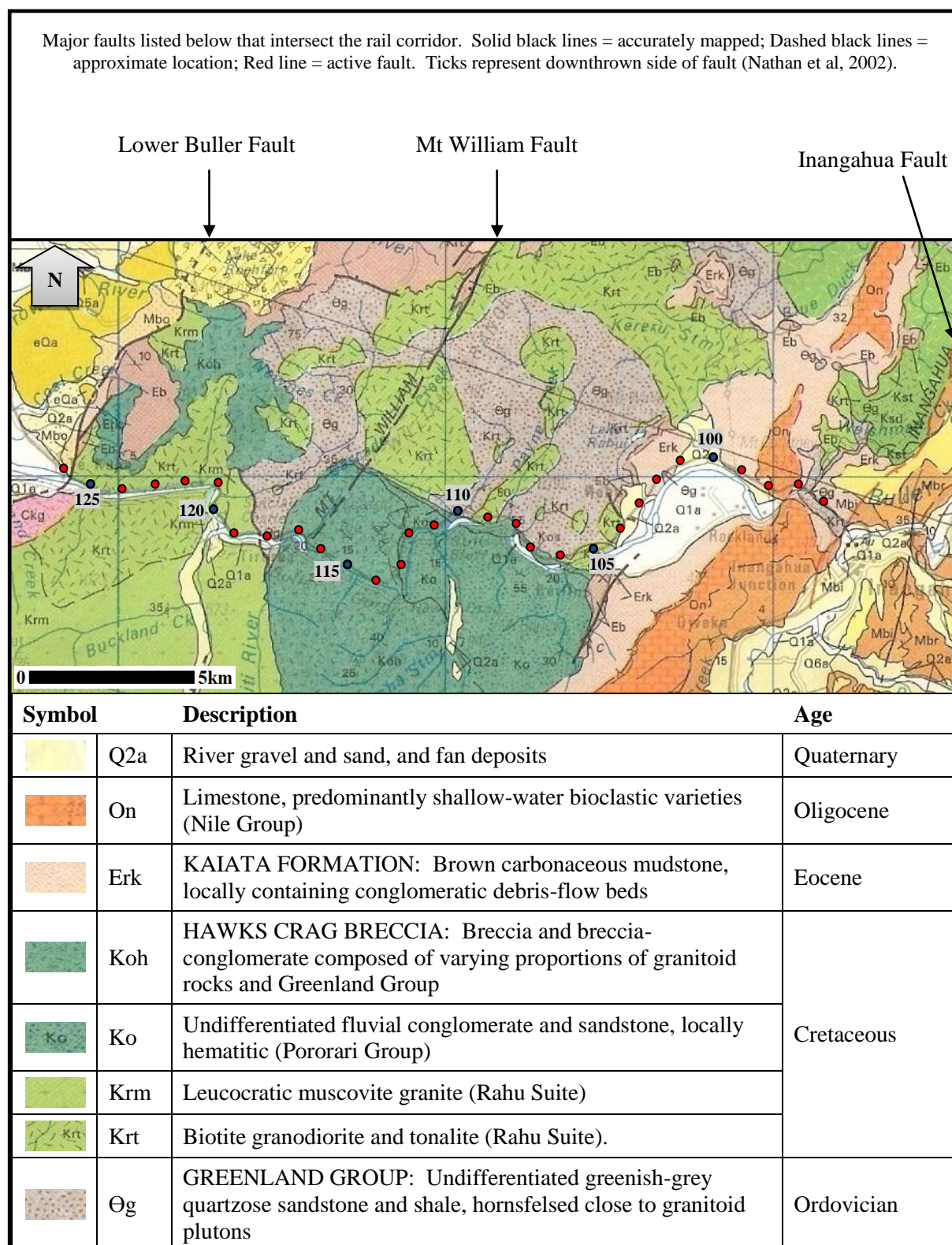


Figure 2.3: Geological map of the project area (based on Bowen, 1964).



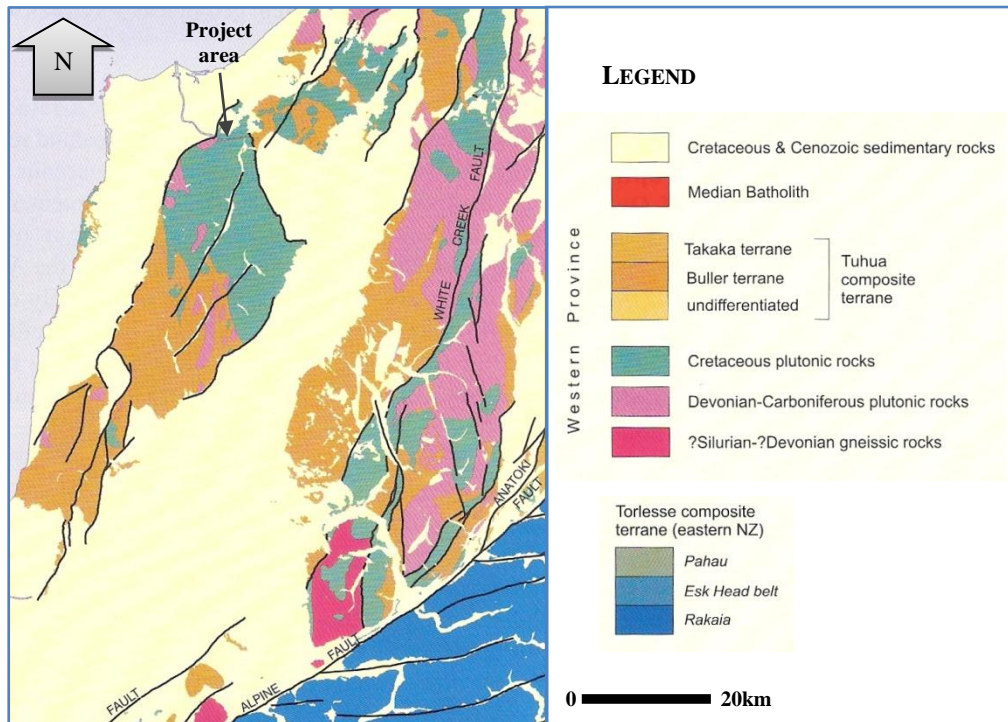
**Figure 2.4: Geological map of the project area (based on Nathan et al, 2002).**

The following summary of geological units present in the project area refers frequently to Figure 2.4, and only those units not incorporated into the legend are represented in the following text with their relevant identifying letters.



### 2.4.1 Basement rocks

Pre-Cenozoic basement rocks in the project area comprise Buller terrane (part of the Tuhua composite terrane); Cretaceous plutonic rocks and Devonian-Carboniferous plutonic rocks (Bowen, 1964 and Nathan et al, 2002). Figure 2.5 shows these basement rocks in the vicinity of the project area, including major fault structures that delineate the units.



**Figure 2.5: Pre-Cenozoic basement rocks of the project area and surrounding region in the South Island, subdivided into tectonostratigraphic terranes (based on Nathan, 2002)**

Within the project area the oldest rocks present (Buller terrane) are Greenland Group sedimentary rocks (Ordovician), comprising interbedded green-grey muddy sandstone (greywacke) and shale (argillite) (Nathan et al, 2002). Greenland Group rocks are predominantly located on the northern side of the Buller River with a thin sliver mapped across the river around SNL118km (Figures 2.3 and 2.4).

Cretaceous granitoids are the next oldest unit, including Rahu Suite plutons of leucocratic muscovite granite (Krm) and biotite granodiorite and tonalite (Krt) (Nathan et al, 2002). These units are mapped on Figure 2.4 between SNL119km and 125km on both sides of the Buller River, and are bounded by the Lower Buller Fault to the west, and an unnamed fault trace to the west. Bowen (1964) maps this sequence as Tuhua Granite (Figure 2.3) at the same metrage locations. Other outcrops of Rahu Suite (krt) are shown on Figure 2.4 at SNL109km and SNL104-105km. Bowen (1964) labels the outcrop at SNL109km as Berlins Porphyry (Figure 2.3).

Early Cretaceous units in the project area comprise Hawks Crag Breccia and Pororari Group coarse-grained, non-marine rocks (Nathan et al, 2002), which are predominant in the project area between SNL110km and 117km). Bowen (1964) shows the fault-bounded contact between Hawks Crag Breccia (Khc) and Ohika Beds (Koh) on Figure 2.3 and this intersects the rail alignment just to the east of the SNL114km metrage peg, trending northwest-southeast.

Hawks Crag Breccia is described by Nathan et al (2002) as a poorly-sorted, matrix-supported breccia and breccia-conglomerate. State Highway 6 (SH6) in the Lower Buller Gorge is cut through Hawks Crag Breccia and a rail tunnel (Tunnel 2) was required through the unit (260m long between SNL113km and 114km). Clasts within the breccia in this area are Greenland Group-derived hornfels, and have been measured up to 500mm in length (Nathan et al, 2002).

### 2.4.2 Tertiary rocks

The oldest Tertiary sedimentary rocks in the region are the Brunner Coal Measures, located to the north of the western extent of the project area (labelled 'Eb' in Figure 2.4, north of SNL123km to 126km). In relation to the project area this unit is not known to directly intersect the rail alignment and is not discussed further.

Eocene sedimentary rocks within the project area include Kaiata Formation. This unit is a massive, dark brown carbonaceous mudstone that contains interbedded mass-flow deposits near Westport (Nathan et al, 2002). Exposures of Kaiata Formation in the rail corridor are intermittently present between SNL99km and 103km (Figures 2.3 and 2.4). Site visits made during this thesis also identified Kaiata Formation sandstones and mudstones around SNL107.50km.

Oligocene sedimentary rocks are particularly relevant to the research project as they incorporate calcareous sediments present at Whitecliffs (Figure 2.4). The depositional history for this end of the project area, and moving more towards the margin of the Murchison Basin, is presented in Chapter 6. The youngest tertiary rocks that intersect the rail alignment are Miocene muddy sandstones (O'Keefe Formation) of the Blue Bottom Group (Nathan et al, 2002). Outcrops of this unit occur only on the western side of the Paparoa Range. O'Keefe Formation (labelled 'Mbo' on Figure 2.4) is present on the western side of the Lower Buller Fault at SNL126km.

### 2.4.3 Quaternary deposits

Quaternary alluvial deposits are mapped in the project area around SNL100km to 101km and SNL103km to 104km. Typical deposits comprise rounded boulders in a sandy matrix (Nathan et al, 2002). While large alluvial fan deposits are often present at the confluence of steep streams that drain range fronts, there are no such features mapped within the Lower Buller Gorge west of the Mackley

River due to the velocity and flow of the Buller River carrying sediment from the surrounding catchments further downstream. The closest mapped alluvial fan deposit is located near the western extent of the project area (Q2a), and north of this unit are weathered (and locally cemented) river gravel and sand (denoted by 'eQa' on Figure 2.4, Nathan et al, 2002). This fan deposit is situated at a remote distance from the Buller River.

#### 2.4.4 Faults

Three main faults are mapped as intersecting the rail: The Lower Buller Fault, Mt William Fault and Inangahua Fault, shown on Figure 2.4. Recent papers by Stirling et al (2002) and Stafford et al (2008) detail the structure and characteristics of these fault lines in regards to the development of seismic hazard models. The Lower Buller Fault typically trends NE-SW, dipping towards the east at approximately 45-60° (Stirling et al, 2002), and bounds the western extend of Tuhua Granite in this section of the rail (between SNL125 and 126km).

The Mt William Fault is inferred to intersect the rail alignment near SNL115.5km. The fault trends NE-SW and dips towards the east at around 45° (Stirling et al, 2002). The Inangahua Fault is mapped to the east of the project area, trending NE-SW and dipping steeply towards the west.

### 2.5 Seismicity

The tectonic setting of the project area presented in Section 2.3.1 shows that the rail corridor is located in a seismically active region, characterised by reverse faults. Seismic activity is a central component to the potential risk of failure at Whitecliffs and other localities in the project area due to the steep topography adjacent to the rail alignment. The two largest magnitude earthquakes that have occurred in close proximity to the project area in the last 100 years are (from Downes, 1995):

- 1929 Buller (Murchison) earthquake – magnitude 7.8M<sub>L</sub> (local magnitude); and
- 1968 Inangahua earthquake – magnitude 7.1M<sub>S</sub> (surface wave magnitude).

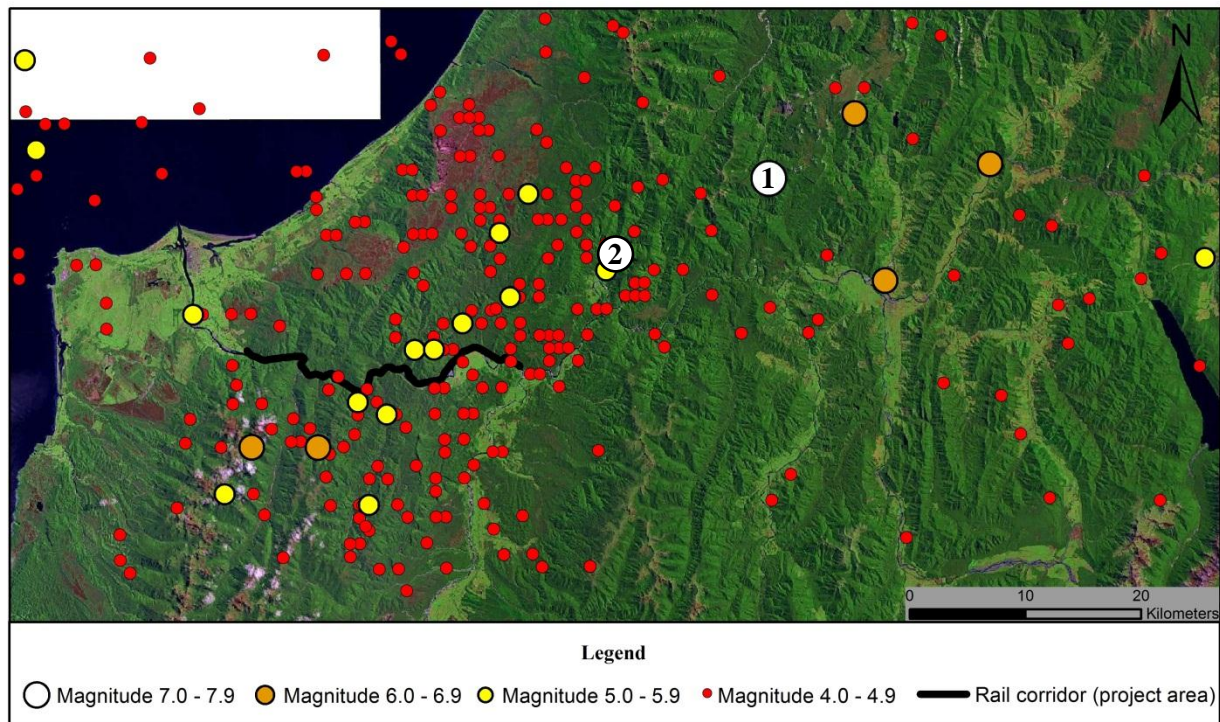
The epicentres for these two events are shown on Figure 2.10. The 1929 event is hereafter generally referred to as the Buller earthquake. This section presents information regarding seismicity relevant to the project area, obtained from GeoNet<sup>2</sup>. Reported impacts to the rail corridor as a result of earthquake-generated landslides, following the Buller and Inangahua events, are presented in Appendix 3.1.

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<sup>2</sup> GeoNet is the name given to a project between the Earthquake Commission and GNS Science that comprises a network of geophysical instruments, automated software applications and staff. The role of GeoNet includes detecting, analysing and responding to geological hazards in New Zealand.

### 2.5.1 GeoNet data

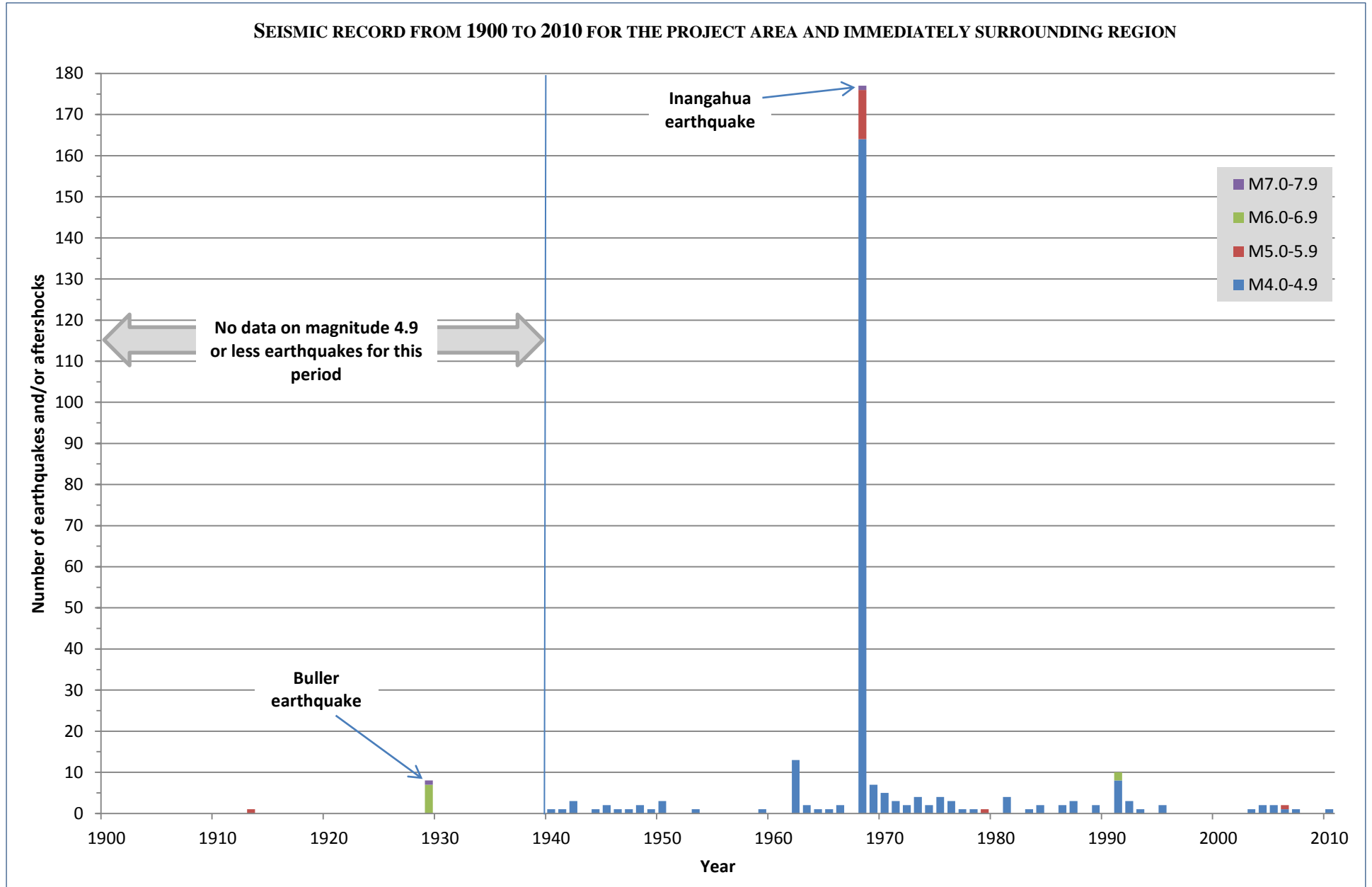
Information available from GeoNet was reviewed to establish recent seismic activity in the project area and immediately surrounding region. Records for earthquakes with a magnitude (M) of M5+ are provided on GeoNet's 'Quake Search' facility from the early 1900s. The historic record for comparatively lower magnitude events (<M5) is available from 1940. It is recognised that various magnitudes have been used historically, and the generic term magnitude is used without specification.



**Figure 2.6: Distribution of magnitude 4.0-4.9 earthquakes (1940-2010) and magnitude 5.0-7.9 earthquakes (1900-2010). 1 = 1929 Buller Earthquake epicentre; 2 = 1968 Inangahua Earthquake epicentre.**

Magnitudes are predominantly recorded as  $M_L$  (only four as  $M_W$ ) in the GeoNet database. A summary of the data provided on Figure 2.10 is discussed below. The year and number of earthquakes recorded between magnitude 4.0 and 7.9 are summarised graphically on Figure 2.7.





**Figure 2.7: Seismic record for magnitude 4.0-7.9 earthquakes between 1900 and 2010 (refer Figure 2.6 for geographical coverage)**

- **Magnitude 5.0-7.9 seismic data:** There were 26 earthquakes of M5 or greater reported on GeoNet between 1900 and 2010. Figure 2.10 shows 22 of these events<sup>3</sup>. All of these earthquakes had a focal depth of <40km, with the exception of the easternmost location shown on Figure 2.6 that had a focal depth of 94km in 2006 (ML = 5.9).
- **Magnitude 4.0-4.9 seismic data:** The distribution of M4.0-4.9 earthquakes is presented on Figures 2.6 for the period 1940 to 2010. Of the 269 M4.0-4.9 earthquakes recorded, only five had a focal depth greater than 40km and are all located near the eastern extent of Figure 2.6.

Figure 2.7 shows that the majority of earthquakes and related aftershocks are associated with the M6.7 Inangahua earthquake in 1968 and M7.8 Murchison earthquake in 1929. The epicentres of these two earthquakes are shown on Figure 2.6. The data record does not include aftershocks from the 1929 event below M5.

## 2.6 Summary

The Lower Buller Gorge project area is characterised by steep topography that is covered with dense native vegetation. High annual rainfall (>2,000mm per year), combined with steep catchments, indicates that the slope adjacent to the rail corridor are susceptible to rainfall-induced landsliding. Seismic data available for the region indicate the Lower Buller Gorge is also located in a seismically active area, characterised by reverse faults. Two large (>M7) earthquakes occurred in close proximity to the eastern extent of the project area in 1929 (Buller) and 1968 (Inangahua).

The geology of the project area is well documented, comprising various geological units, including basement sedimentary rocks (Greenland Group), granites and sedimentary breccias (Hawks Crag Breccia). Tertiary sedimentary rocks are present at the eastern end of the project area, including limestone that forms the outcrop at Whitecliffs, around SNL97.5km. Quaternary alluvial deposits, sourced from the Buller River and its catchment, form the most recent geological units.

To characterise landslide susceptibility and hazards, the historical record requires review to determine previous slope areas that have failed along the rail corridor; the relevant triggering mechanism, and future susceptibility. The landslide inventory developed is fully documented in Chapter 3.

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<sup>3</sup> One event in 1929 did not have a magnitude specified and three events were reported with the same coordinate reference (two in 1929 and one in 1968).

## CHAPTER 3: LANDSLIDE RISK MANAGEMENT

### 3.1 Introduction

Development of a landslide inventory is a key component to assessing the geotechnical risk of slope failures within the Lower Buller Gorge rail corridor, including the permanent speed restriction areas at Whitecliffs and Te Kuha. AGS (2007) define a landslide inventory as: *‘an inventory of the location, classification, volume, activity and date of occurrence of individual landslides in an area’*. The objective of Chapter 3 is to collate all available historical information related to the rail construction and landslide occurrences for the purpose of:

- Risk identification (landslide inventory);
- Establishing information gaps; and
- Understanding and outlining the thesis limitations.

In addition to the information requirements outlined by AGS (2007), the triggering mechanism that initiated slope movement is also important. The research hypothesis of a seismic trigger being the most likely source for a large-scale failure at Whitecliffs, and other locations within the project area, requires a review of available information to determine whether there is any evidence to suggest previous failures directly attributable to seismic shaking.

A desktop study was conducted to achieve the above objectives. Information for the desktop study was obtained from KiwiRail, and a variety of other sources, including those outlined in Table 3.1. Sections 3.2 and 3.3 provide detailed background information relating to the rail construction history, and the timing, and rationale, behind the permanent speed restrictions at Whitecliffs and Te Kuha, based on literature reviews and discussions with key railway personnel. All information not sourced directly from KiwiRail (Section 3.4) is provided in Appendices 3.1 to 3.4.

The main outcome of the desktop study has been the development of a landslide inventory, presented in Section 3.5. This inventory is applied further in Chapter 4 in terms of landslide susceptibility mapping for the project area and identification of key risk sites.

Details regarding landslide occurrences adjacent to SH6 are included in the desktop study, where relevant, as a result of high intensity and/or long duration rainfall events. The inclusion of these records is considered relevant in terms of establishing landslide frequency since impacts to transport corridors would rarely be constrained to one side of the gorge only, despite the absence of direct reports specific to the rail corridor in many cases.

Table 3.1: Desktop study items and information sources

Item		Information source(s) and summary of detail obtained	Thesis section
Background	<b>Rail construction history</b>	A review of the publication by the NZRLSI (1964) provided detailed information regarding construction of the Lower Buller Gorge rail corridor.	3.2
	<b>Permanent speed restrictions</b>	Discussions were had with key railway personnel to establish whether the rationale and timing behind the permanent 25km/hour speed restrictions at Whitecliffs and Te Kuha is known.	3.3
Landslide inventory data sources	<b>KiwiRail information</b>	Review of data and reports provided by KiwiRail relevant to landslides, derailments and track inspections, including discussions with key railway personnel.	3.4
	<b>West Coast Regional Council records</b>	Collation of relevant entries reported in the WCRC natural hazards review document (DTec Consulting Limited, 2002) to establish the frequency of reported flooding and landslide incidents relevant to the Lower Buller Gorge, including the triggering mechanism.	A3.1*
	<b>Aerial photography</b>	Stereoscopic viewing and interpretation of historic aerial photography flown in 1946, 1959 and 1985 from NZAM. Review of aerial imagery available from Google Earth and MapToaster.	A3.2*
	<b>1929 Buller earthquake</b>	Literature reviews conducted regarding landslide occurrence in the region as a result of the magnitude 7.8 ( $M_s$ ) earthquake.	A3.3*
	<b>1968 Inangahua earthquake</b>	Literature reviews conducted regarding impacts to the rail corridor and landslide occurrence as a result of the magnitude 7.1 ( $M_s$ ) earthquake. Landslide occurrence was also established through a search of information held in the Inangahua Earthquake Museum, and a review of aerial photography flown in 1970.	A3.4*
<b>Final landslide inventory</b>		Collation of all data sources available to produce a landslide inventory for the project area, including triggering mechanisms (where available).	3.5
*A = Appendix 3 references			

## 3.2 Rail construction history

The SNL was formerly known as the Stillwater to Westport Line (SWL). This changed to SNL in 2009 to incorporate the Ngakawau branch line<sup>4</sup>. Construction of rail within the project area occurred over a 37 year period between 1906 and 1943 (NZRLSI, 1964). A 9km length of rail between Westport and Te Kuha was initially opened in 1912, while the operation of trains through the entire Lower Buller Gorge was only officially opened in 1943. The following timeline provides greater detail regarding the rail corridor construction history. The timeline and subsequent discussion is based on information from NZRLSI (1946):

- 1885:** Minister for Public Works advises the Government proposes to construct a branch line between Westport and Inangahua to connect this section with the main line.
- 1906:** First steps in rail construction undertaken following surveys that identified the Buller Gorge as the most suitable link between Westport and Inangahua.
- 1910:** The section of line between Westport and Te Kuha is completed. Flax and timber were the predominant supplies being transported at this stage.
- 1915:** Construction halted due to World War I.
- 1926:** Formation of the rail from the Inangahua (eastern) end of the corridor commenced. Construction also started again from western end due to the formation of the Westport-Cascade Coal Company Ltd, which had a rail connection at Cascade Creek.
- 1927:** The first train load of coal from Cascade Creek departed on 20 July 1927. A siding at Cascade Creek contained coal bins that received the product via approximately 12km of water races that sluiced the coal through flumes from the mine entrance.
- 1928:** Hawks Crag is reached. Progress is typically slow due to the requirement for: excavations into rock; filling of large areas (embankments); culvert, bridge and tunnel construction.
- 1932:** Construction halted at both ends of rail corridor in response to a pending world depression. The report noted that foundations for a curved viaduct over Cascade Creek had been completed by 1932, but no specific date was provided.
- 1936:** Construction work resumed. Despite the advances in machinery and tools available, the landscape and rainfall made progress slow.
- 1939:** Reduced machinery and workers available at the start of World War II.
- 1941:** Final spike was driven at Slaty Creek (approximately 29km from Westport).
- 1942:** Rail is operational for transportation of supplies after track formation has been stabilised and cuttings completed. Vulcan Railcars began operating between Westport and Greymouth on 7 September 1942.
- 1943:** Official opening date of the rail between Te Kuha and Inangahua. Formally handed over to the Railways Department.

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<sup>4</sup> <http://www.linz.govt.nz/placenames/consultation-decisions/a-to-z/railway-lines/report.aspx>

Stations listed in the NZRLSI (1964) report, from east to west along the rail corridor, were: Inangahua; Buller; Mackley Ballast Pit; Rahui; Tiroroa; Cascade; Te Kuha; New Zealand Cement Company Siding; Westport; and Queen Street. It was noted that the stations at Te Kuha and Cascade had already been closed by 1964.

Rainfall was highlighted in the report as a constant challenge to construction. An example provided from 1938 was the occurrence of over 5,000mm of rainfall recorded within 176 days. The report also commented that despite ‘high speeds’ not being possible due to the curvature of the line, the track was laid to modern standards, including automatic signalling. The frequency of train movements in the 1960s through the Lower Buller Gorge was typically one or two goods train per day (each way), and a twice daily railcar service with travel times of 1.5 hours and 50 minutes respectively.

### 3.3 Permanent speed restrictions

To assess the risk of removing the existing 25km/hour speed restrictions at Whitecliffs and Te Kuha, the rationale and timing of when the restrictions were imposed needs to be understood. KiwiRail provided contact information for current and former rail personnel that had knowledge of the history and operations within the project area. The objectives of the discussions were to ascertain:

- A. Timing:** When were the two 25km/hour speed restrictions imposed.
- B. Rationale:** Why the speed restrictions were implemented and was there justification for 25km/hour.
- C. Impacts:** Previous direct impacts to the rail corridor at Whitecliffs and Te Kuha due to slope movement.

The template used for the discussions is provided on the following page. Questions asked were all specific to rail. Table 3.2 summarises the outcome of the personal communications in terms of the detail provided specific to Whitecliffs and Te Kuha. General information obtained during the discussions regarding slope movement hazards within the rail corridor is summarised in Section 3.4.4.

**Table 3.2: Summary of information available regarding permanent speed restrictions at Whitecliffs and Te Kuha**

Objective	Whitecliffs	Te Kuha
A: Timing	Late 1960s to early 1970s.	Late 1980s (indicative)
B: Rationale	Unknown	Unknown
C: Impacts	Infrequent*	Frequent (around once every two years) Late 1980s – rainfall induced failure
*One small-scale incident recalled around 2005 (limestone blocks with a total volume <1m <sup>3</sup> ).		

University of Canterbury – HAZM thesis (Kristel Franklin)  
SWL 96km – 126km

#### Interview record sheet

DATE:	17 March 2010
NAME:	Brent Lancaster
LOCATION:	Midas Place, Christchurch.

#### Stillwater – Westport Line

Existing speed restriction of 25km/hr at:

White Cliffs: between 97.51 and 97.53km

Te Kuha: between 124.05 and 124.50km

#### Questions

The purpose of this interview is to understand the background relating to the following areas of interest, or other personnel that may be able to provide relevant information:

- The existing speed restriction:
  - When was it imposed?
  - Why 25km/hr?
- Rockfall hazards:
  - Records of incidents of any scale within the section of railway above?
  - Any known earthquake-triggered rockfall incidents known of nationwide?
- Other hazards (flood, landslides):
  - Records of incidents associated with natural hazards of any scale within the section of railway above?
- Early warning systems relating specifically to rail and natural hazards:
  - Are there any early warning systems existing or proposed within New Zealand?
  - Economic viability for the SWL of installing an early warning system?

**Thank you for your time!**

There is uncertainty regarding the date the speed restrictions were imposed, as indicated in Table 3.2. Background information regarding the dates shown is discussed further in Sections 3.3.1 and 3.3.2. There is no information available, or prior knowledge, that provided any clarification of why 25km/hour was specified. The maximum allowable speed through the two sections is considered to be 40-50km/hour based on track curvature and limited visibility at Whitecliffs and Te Kuha.

Discussions to date with railway personnel indicate that people with knowledge of the area accept that the physical environment poses continued operational and maintenance challenges in the Lower Buller Gorge. The removal of speed restrictions at Whitecliffs and Te Kuha was not considered a major concern provided that improved monitoring procedures at Whitecliffs were sufficient to enable early warning of a major rockfall event.

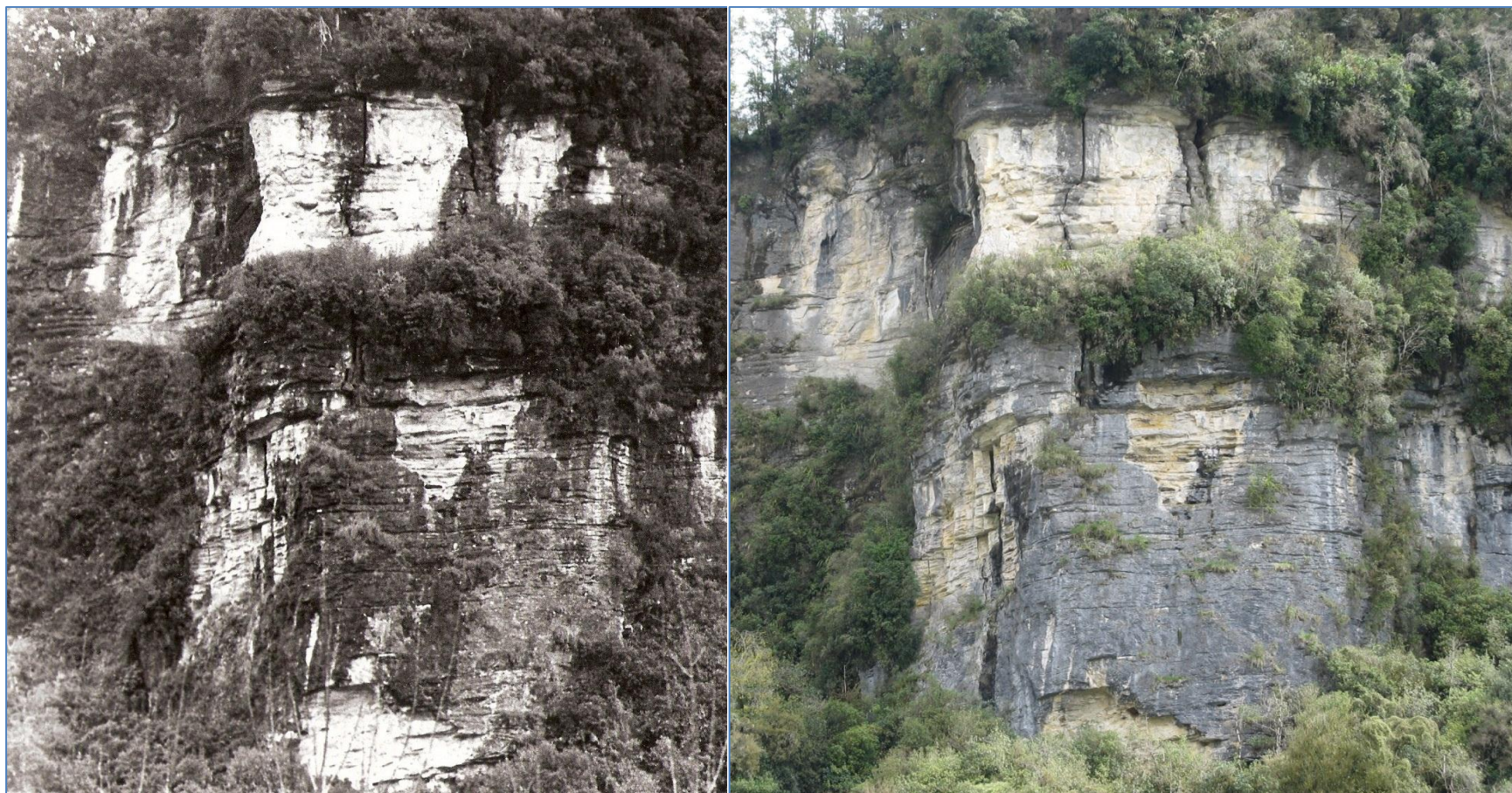
### 3.3.1 Whitecliffs

An important finding of the conversations was that there has been no recollection of a rockfall originating from Whitecliffs of sufficient volume to cause major rail disruptions. One small-scale incident of rocks falling onto the rail was recalled to have occurred around 2005 (H. Armstrong, personal communication, November 2010). This event did not cause a derailment or major disruption to rail operations (total volume  $<1\text{m}^3$ ). This suggests that the talus apron that has formed at the base of Whitecliffs retains the majority of blocks that may topple or fall from the cliff-face.

At a minimum, the speed restriction at Whitecliffs has been in place since the 1980s (B. Lancaster, personal communication, March 2010). This date was based on the opinion that the potential for failure at Whitecliffs was identified as a risk after a resurgence in coal production in the mid to late-1980s, and the introduction of 70 tonne wagons (laden) being transported through the Lower Buller Gorge. Another opinion was that the restriction at Whitecliffs has been in place since at least 1968, after the magnitude 7.1 Inangahua Earthquake on 24 May 1968 (H. Armstrong, personal communication, November 2010). It is noted that collapse of the limestone cliffs on the southern side of the Buller River resulted in loss of life and significant damage to road infrastructure during this event.

A series of photographs taken of Whitecliffs between 1975 and 1981 were provided by KiwiRail for review. The photographs were taken from the same location on each occasion to determine any discernible changes in the cliff-face profile over time. Examples of the photographs are shown in Figures 3.1 and 3.2 in comparison to the profile evident in 2011. From the perspective shown in Figures 3.1 and 3.2, there are no visible changes in the cliff-face profile between 1975 and 2011. The focus on Whitecliffs evident at this time (from the mid-1970s) indicates awareness of the potential rockfall hazard, and corresponds to the speed restrictions being implemented prior to the 1980s. It is unknown why this photographic monitoring technique was not continued after 1981.





**Figure 3.1: Whitecliffs profile comparing 1979 (left) and 2011 (right) photographs**





**Figure 3.2: Defects in near vertical limestone outcrop at Whitecliffs in 1976 (left) and 2011 (right).**

### 3.3.2 Te Kuha

Background knowledge regarding the speed restriction at Te Kuha is less certain than for Whitecliffs. It is considered that it was likely imposed in 1987 due to rainfall over a six day period (14 to 20 January 1987) that caused widespread disruption to transportation routes throughout the region (B. Lancaster, personal communication, March 2010). Damage during this event specific to the rail corridor included a large landslide that destroyed half of the Windy Point rail tunnel (Tunnel 7) near Te Kuha. This tunnel was subsequently daylighted.

The area in general around Te Kuha between SNL124 and 126km is frequently regarded by rail personnel as one of concern in terms of landslides, with a high frequency (about one every two years) of debris impacting the rail corridor. No details on volumes or run-out distances were provided.

## 3.4 KiwiRail information

Section 3.4.1 summarises information provided by KiwiRail in regards to reported incidents between SNL96 and 126km that involved landslides, including any derailments directly attributable to movement from upslope sources. Sites listed by PGL (2007) relating to slope instability features are discussed in Section 3.4.2. One site-specific report for a landslide in 2006 at SNL112.96km is presented in Section 3.4.3. General information from various discussions with KiwiRail and Ontrack staff is summarised in Section 3.4.4.

### 3.4.1 Landslide data

Information was requested from KiwiRail regarding reported landslides, flooding events and derailments on the SNL. Landslides are generally referred to as ‘slips’ in the KiwiRail data sets. The historic record provided was based on relatively recent data. For landslide and flooding incidents that impacted rail operations, data was provided for the period 1 July 2004 to 5 April 2011. For derailments that occurred due to landslides, data was provided for the period 1 August 2006 to 27 April 2011.

A review of the information provided by KiwiRail indicated that 503 incidents relating to slips and flooding were reported nationally between 1 July 2004 and 5 April 2011. Of these, 28 occurred on the SNL, including 12 within the project area between SNL96km and 126km. Only one recorded derailment occurred on the SNL due to a slip during the period 1 August 2006 to 27 April 2011 (out of nine reported nationally). The incident occurred at SNL105.60km. A summary of relevant information provided by KiwiRail is provided in Table 3.3, including the incident date, SNL metrage and description.

**Table 3.3: Flooding, landslide and derailment incident reporting during the period 1 July 2004 to 5 April 2011 (based on KiwiRail data)**

Date	Location (metrage on the SNL)	Description
17 February 2005	105.30	Digger dispatched to site. Services suspended pending slip clearance.
7 March 2005	107.50	Slip reported by AC Transfield Services. Suspected that service 841 struck small slip, and the Train Controller imposed a 10kmh speed restriction.
25 March 2005	113.00	846 struck rock on track between Tiroroa - Berlins, no damage. AC Transfield Services completed inspection, and track clearance was received at 1845 hours.
3 June 2005	118.50	Locomotive Engineer reported removing a "large bush". No damage reported. AC Transfield Services imposed a 10kmh speed restriction until track inspection completed.
18 January 2006	119.00	Slip in the Buller Gorge located by wet weather track inspection.
23 April 2006	104.85	845 struck slip in the Buller Gorge. Locomotive Engineer made an emergency brake application travelling on 25km/hour speed restriction and reported a collision with a small slip and tree with further movement noted on the above hillside. A severe weather warning was in force at the time.
30 July 2008	124.00	Slip in the Buller Gorge. 124.00 - 124.50km line closed (Te Kuha).
31 July 2008	123.80	Tree reported across track at 123.80km. Ganger doing run through Buller Gorge, after clearing slip, came across it. High-rail digger to assist, estimated track clearance at 1140 hours.
10 January 2010	119.00	Slip in the Buller Gorge at 119.00km found by gang on wet weather run.
4 August 2010	120.00	Large boulder landed on, and damaged, one rail.
13 August 2010	112.00	845 struck a tree 112.00km SNL. Track was checked by Westport ganger and all clear given at 1100 hours. No damage to locomotive 845, no delays.
13 January 2011	105.60	846 derailed at 105.60km on SNL between Inangahua and Tiroroa. Derailed five wagons.
6 March 2011	125.50	Work Train 82 struck a slip at approximately 125.50km SNL between Tiroroa and Westport. Slip about 2 to 3 metres above rail level. Damage to cowcatcher fuel tank punctured.

The reported incidents on the SNL do not include details regarding volume of material or the nature of failure. It is recommended that future reporting attempts to characterise the nature of the landslides, including a photographic record, and basic details regarding volume of soil/rock released and dimensions of the area impacted.

The occurrence of trees and vegetation slipping onto the rail is highlighted in the reported incidents. Of the thirteen entries in Table 3.3, four refer specifically to trees on the rail (3 June 2005, 23 April 2006, 31 July 2008 and 13 August 2010). Two reports refer to possible rockfalls (25 March 2005 and 4 August 2010). There were no reported incidents of rockfalls in the vicinity of Whitecliffs. Two of the entries are related to known slope stability issues around Te Kuha (SNL124.00km on 30 July 2008 and SNL125.50km on 6 March 2011).

Information from an additional KiwiRail database for a similar period (February 2004 to June 2011) included entries not recorded in Table 3.3. The additional incidents are presented in Table 3.4, but only limited detail was provided.

**Table 3.4: Landslide related incident reporting during the period February 2004 to June 2011**

Date	Location (SNL metrage)	Description
28 February 2004	Not stated	Slip-derailment Westport.
22 November 2006	Not stated	Slip in the Buller Gorge.
6 August 2007	102.80km	Possible slip.
24 November 2008	97.50km	Bad slip at Whitecliffs.
17 April 2009	Cascade	Surface flooding.
19 December 2009	120.50km	Flooding at Cascade.
13 September 2010	Not stated	Small rockfall in Buller Gorge.
23 April 2011	125.80km	Slip.
24 April 2011	123.10km	Slip.

### 3.4.2 Geotechnical assessment (PGL, 2007)

The PGL (2007) report discussed in Chapter 1, Section 1.8, included an itemised list of sites labelled ‘issues’ that required remediation. The following list is based on sites that PGL (2007) identified as being of concern in terms of slope stability (labelled by SNL metrage and a brief description based on text by PGL). Interpretations based on the PGL (2007) photographic record are in brackets, where relevant.

- 97.60 Whitecliffs, rockfall potential
- 101.65 Slip (shallow regolith failure)
- 104.80 Slip blocked water table drain (surficial material only, <1m depth)
- 105.05 Slip (surficial material only, <1m depth)
- 106.65 Large historic landslide
- 107.55 Large historic landslide, including derailment in 2004
- 110.95 'Historic landslip' (photograph appears recent, small-scale shallow regolith failure)
- 111.57 Historic landslip (re-vegetated)
- 111.63 Slip on batter above track (shallow failure)
- 112.92 Historic landslide (shallow failure, partially re-vegetated)
- 115.95 Active landslide (shallow failure, slope angle <20°)
- 116.00 Active slip (shallow failure, slope angle <20°)
- 118.86 Sinclair Castle (historic rockfall site)
- 119.72 Rockfall potential
- 124.27 Historic rockfall site, western portal of Tunnel 6
- 125.40 Large historic landslide

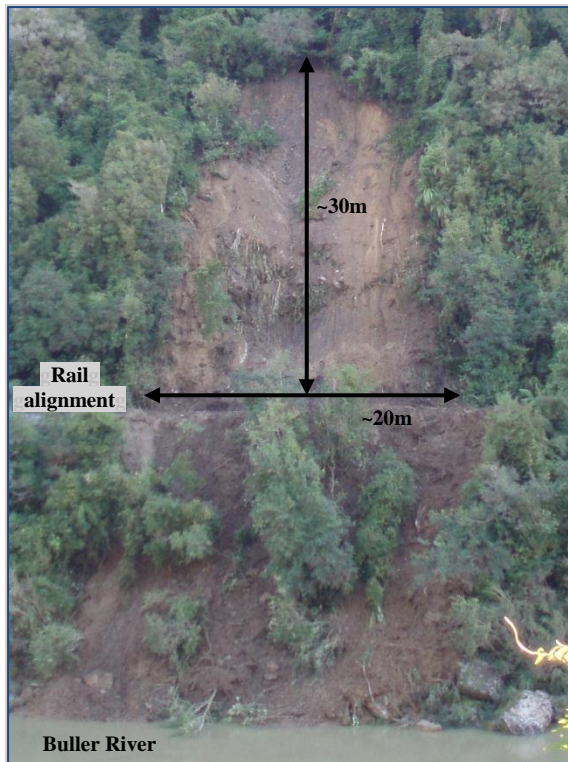
The terminology adopted by PGL (2007) is variable and little or no detail was provided on landslide dimensions, volumes, mechanism of failure or the triggering event. Photographs presented by PGL (2007) do give an indication of the landslide feature scale, which is incorporated into the landslide inventory, where relevant, in Section 3.5.

### 3.4.3 SNL112.96km landslide report (Ontrack, 2006)

An initial report provided by KiwiRail relating to a landslide that occurred at SNL112.96km was reviewed (Ontrack, 2006). The geology at this location comprises Hawks Crag Breccia (refer Chapter 2, Figures 2.3 and 2.4). The shallow landslide had dimensions of approximately 20m (width) by 30m (height), as indicated in Figure 3.3.

This event closed the SNL for three days (12-15 June 2006). Slope angles were estimated at near vertical (over-steepened) from the rail alignment up to 10m, reducing to 40-50° from 10m height to the headscarp. A volume estimate was not given but due to the shallow (<2m) failure surface approximately 100m<sup>3</sup> of material was released.





**Figure 3.3: View of landslide at SNL112.96km from State Highway 6 (based on Ontrack, 2006).**

Rainfall was the triggering event, which also caused an approximate  $20,000\text{m}^3$  landslide adjacent to SH6, resulting in a road closure for three days (Ontrack, 2006). The main conclusions from the initial report are summarised below:

- The change in slope angle at 10m above the rail alignment was interpreted as delineating the change from a cut slope to natural slope.
- A loose boulder ( $>3\text{m}$  in diameter) was noted precariously positioned a few meters above the landslide headscarp.
- The mode of failure was interpreted as a shallow translational slide.
- No evidence to suggest further regression of the headscarp will occur, based on previous similar types of failures noted around this location that have re-vegetated naturally without on-going stability issues.
- A detailed investigation was recommended to determine the most appropriate long-term management strategy for the landslide.

General comments made in the report relate to the on-going challenges faced in regards to small to medium-scale slope stability issues throughout the Lower Buller Gorge rail corridor, and the associated difficulty in prediction or mitigation of geotechnical hazards. The closing comment highlighted a recommendation for a dedicated excavator (high-rail or heavy-duty trolley) for rapid response, in the event of landslides in the Lower Buller Gorge, to improve general route security.

This landslide does not appear in Table 3.3 or 3.4. Previously reported events have occurred in close proximity to the 2006 landslide at SNL112.96km, including 13 August 2010 at SNL112km, and 25 March 2005 at SNL113km (refer Table 3.3).

#### 3.4.4 General information

During discussions held with rail personnel, as outlined in Section 3.3, a question was asked regarding details on known historic rockfall or other slope movement hazards in the project area. Three locations additional to Whitecliffs and Te Kuha were highlighted, as outlined below:

- **SNL107.50km:** Site of large landslide that caused a train derailment in 2004 (PGL, 2007). This event is not recorded in Table 3.3 or 3.4.
- **SNL118.50km:** Sinclair Castle – on-going slope stability issues, including a landslide that closed the rail for a few hours in January 2006. This event is recorded in Table 3.3 at SNL119.00km.
- **SNL121.00km:** Cascade – numerous rock avalanches, particularly in the late 1980s.

Site specific details from these sites obtained during field work conducted between 2009 and 2011 are presented in Chapter 5 and Appendix 5.1. Problems associated with loss of ballast, retaining structures and erosion from the Buller River were brought up by KiwiRail personnel to highlight that line integrity is often more at risk, in terms of safety, time and cost, when material beneath the railway line is compromised, rather than material falling onto the line from above. The importance of adequate drainage design, ballast and retaining structures is recognised but does not form a part of this thesis. These features have been investigated and reported previously by PGL (2007).

### 3.5 Final landslide inventory

The desktop study has identified a number of historic and comparatively more recent landslide features of varying scales in the project area that have impacted the rail corridor. This section presents the final inventory (hazard identification) that will be used for analysis and further evaluation in Chapter 4 (Landslide Susceptibility). The following discussion outlines the information gaps and associated limitations of the thesis based on the available historic record of landslide occurrence.

The predominant landslide triggering mechanism in the Lower Buller Gorge is rainfall. Earthquake-generated landslides have been recorded primarily as a result of the 1968 Inangahua Earthquake. Limited detail was available for landslide occurrence in the Lower Buller Gorge after the 1929 Murchison earthquake since the rail corridor was not fully completed at this time (Appendix 3.3 and 3.4).

Specific details regarding landslide volumes, mechanism of failure, run-out distance and remedial works are very limited. Table 3.5 summarises all available information from the various sources outlined in this chapter that had a specific metrage location. The majority of events listed have directly impacted the rail corridor and operations. The exception is a few entries by PGL (2007) that did not necessarily disrupt rail operations either due to the small volume of material released or comparatively low slope angles ( $<20^\circ$ ), which combined with distance from the rail, would not likely have caused any impacts aside from blocking culverts. Specific events that resulted in an impact with a train and/or a derailment are represented by ‘T’ and ‘D’ respectively in the volume estimate column. Slope movement features identified from the aerial photograph review at locations remote from the rail alignment were not incorporated into the final inventory.

The data presented is in order of distance, from SNL96km to 126km. A full inventory would encompass hundreds of rainfall-induced failures of varying sizes, many of which would have minimal impact on the railway operations, and the data presented in Table 3.5 represents only a fraction of actual occurrences. The triggering mechanism, information source(s), and date of occurrence, where known, are also noted. Given the absence of accurate volume measurements, a logarithmic scale from  $<10\text{m}^3$  (very small) to  $>10,000\text{m}^3$  (very large) has been adopted to provide a comparison of the event scale, as indicated below:

- $<10\text{m}^3$                       Very small
- $10\text{-}100\text{m}^3$                       Small
- $100\text{-}1,000\text{m}^3$                       Medium
- $1,000\text{-}10,000\text{m}^3$                       Large
- $>10,000\text{m}^3$                       Very large

In cases where ‘slip’ is the only reference made to a landslide, and there are no photographs, it is not possible to assign a volume estimate (denoted by ‘unknown’ in Table 3.5). Landslide travel distances and velocity are not shown on Table 3.5 due to the absence of this information in the historic record. The rail was formed onto the side of natural hillslopes at many locations in the Lower Buller Gorge, rather than by cutting and/or benching, and the distance between the base of slope and track is frequently  $<2\text{m}$  horizontally. Based on these track formation characteristics, the travel velocity and distance factors are less variable components, in terms of direct impacts to rail, as slope failures originating on steep slopes (typically  $\geq 20^\circ$ ) above to the corridor are likely to intersect the track regardless, with the exception of some smaller-scale landslides. The situation at Whitecliffs differs from this generalised scenario due to the established talus apron at the base of the cliffs that effectively captures the majority of blocks from the outcrop.

**Table 3.5: Final landslide inventory for the project area**

Metrage (SNLkm)	Volume Estimate	Triggering Mechanism(s)	Event Date	Information Source
95.96	Unknown (assumed at least medium)	Earthquake	24 May 1968	IEM
96.00	Various – small to medium	Rain, earthquake	Various	API
~97.50	Very small (rockfall)	Unknown	2005	KiwiRail
97.50	Unknown	Unknown	24 November 2008	KiwiRail
97.74	Unknown	Earthquake	24 May 1968	Duckworth (1968)
98.00	Small (rockfall)	Earthquake	May 1968 (assumed)	API
101.65	Very small to small	Rain	Pre-2007	PGL
102.80	Unknown	Rain	6 August 2007	KiwiRail
104.80	Very small	Rain	Pre-2007	PGL
104.85	T – Small	Rain	23 April 2006	KiwiRail
105.05	Very small	Rain	Pre-2007	PGL
105.30	Unknown	Rain	17 February 2005	KiwiRail
105.60	TD – Unknown	Rain	13 January 2011	KiwiRail
106.65	Medium to large	Unknown	Unknown	PGL
107.50	Small to large	Rain	Various	KiwiRail, PGL
107.50	Very small	Rain	7 March 2005	KiwiRail
107.55	T – Medium to large	Rain	2004	PGL
108.60	Small to medium	Unknown	Pre-1985	API
108.63	Medium to large	Earthquake	24 May 1968	Duckworth (1968)
110.95	Small	Rain	Pre-2007	PGL
111.00	Small	Unknown	Pre-1985	API
111.04	Unknown	Earthquake	24 May 1968	IEM
111.57	Small to medium	Unknown	Pre-2007	PGL
111.63	Small to medium	Unknown	Pre-2007	PGL
112.00	T – Very small (tree)	Rain	13 August 2010	KiwiRail
112.92	Small to medium	Rain	Pre-2007	PGL
112.96	Small to medium	Rain	12 June 2006	KiwiRail
113.00	T – Very small	Rain	25 March 2005	KiwiRail

Metrage (SNLkm)	Volume Estimate	Triggering Mechanism(s)	Event Date	Information Source
113.00	Various	Unknown	Pre-1947	API
113.80	Large	Unknown	Pre-1947	API
114.00	Large to very large	Unknown	Pre-1947	API
115.95	Small	Rail	Pre-2007	PGL
116.00	TD – Unknown	Rain	10 February 1944	WCRC
116.00	Very small	Rain	Pre-2007	PGL
116.50	Small	Unknown	Pre-1959	API
116.68	Unknown	Earthquake	24 May 1968	IEM
117.28	Unknown	Earthquake	24 May 1968	IEM
117.44	Unknown	Earthquake	24 May 1968	IEM
117.48	Large to very large	Earthquake	24 May 1968	IEM, Duckworth (1986)
118.50	Very small (vegetation)	Rain	3 June 2005	KiwiRail
118.80 to 119.50	Very large	Unknown	Pre-1947	API
118.86	Small to large	Rain, Earthquake	Various	KiwiRail, PGL
119.00	Unknown (rail closed)	Rain	25 March 1964	WCRC
119.00	Small to medium	Rain	Pre-1985	API
119.00	Unknown (rail closed)	Rain	18 January 2006	KiwiRail
119.00	Unknown	Rain	10 January 2010	KiwiRail
120.00	Very small (rockfall)	Rain	4 August 2010	KiwiRail
120.00	Small to medium	Unknown	Pre-1985	API
121.00	Small to large	Rain	Various	KiwiRail
121.50	Various (small to large)	Unknown	Various	API
122.43	Small to medium	Earthquake	May 1968	IEM
123.10	Unknown	Rain	23 April 2011	KiwiRail
123.80	Very small (tree)	Rain	31 July 2008	KiwiRail
124.00	Small to medium (track closed)	Rain	30 July 2008	KiwiRail
124.27	Large	Rain	Pre-2007	PGL

Metrage (SNLkm)	Volume Estimate	Triggering Mechanism(s)	Event Date	Information Source
124.40	Medium	Unknown	Between 1985 and 2001	API
125.00	Large	Rain	14 January 1987	WCRC
125.40	Very large	Unknown	Pre-2007	PGL
125.50	T - Small	Rain	6 March 2011	KiwiRail
125.80	Unknown	Rain	23 April 2011	KiwiRail
<b>Volume estimate key:</b> T = train struck landslide D = derailment		<b>Information source key:</b> PGL = Pacific Geotech Limited (2007) API = Aerial Photograph Interpretation WCRC = West Coast Regional Council IEM = Inangahua Earthquake Museum		

AGS (2007) also included landslide classification as an important component of a landslide inventory. The dominant mechanism of failure in the Lower Buller Gorge is shallow translational slides, comprising weathered bedrock, soil and vegetation. Rock avalanches with source material originating from catchments many hundreds of metres above the rail alignment occur at Cascade and Sinclair Castle. Rockfalls and topples are typical mechanisms of failure at Whitecliffs and other locations that occur in over-steepened rock outcrops (defect-controlled failures).

### 3.6 Summary and synthesis

Challenges related to steep topography, geological characteristics, high annual rainfall and access limitations within the Lower Buller Gorge were encountered during the construction of the rail corridor between 1912 and 1942, in addition to economic and societal influences. These challenges still exist for the effective on-going operation of the SNL today.

Existing speed restrictions at Te Kuha and Whitecliffs indicates awareness by rail operators of the possible slope movement hazards at these two locations. There is very limited information available for the timing and rationale behind the speed restrictions.

The purpose of landslide inventory development (Table 3.5) is risk identification as it highlights specific locations in the project area that have been impacted by slope movement originating from sources above the rail corridor. Based on the research conducted regarding landslide occurrence, and triggering mechanisms, information gaps are present in terms of:

- Temporal data prior to 2004;
- Failure mechanisms (classification);



- Landslide volumes, and material; and
- Run-out distances.

Despite the limitations the inventory developed provides a basis for analysis. Collation of landslide occurrences from all sources listed in Table 3.5 enables the most robust assessment currently possible for the project area in terms of documented slope failures.

It is recommended that future reporting should characterise the nature of the landslides, including a photographic record, and basic details regarding volume of soil/rock released and dimensions of the area impacted. The initial report for the landslide at SNL112.96km in 2006 provides a good example of the level of detail possible and even an abbreviated version of this type of report would be sufficient for establishing a more robust database.

Detailed interpretation of the data provided in this chapter is presented in Chapter 4, including spatial representation of the landslide occurrences using ArcGIS as a platform for data presentation, and correlation to slope angles and geology characteristics of the project area. The ‘hazard’ component of the Landslide Risk Management framework is outlined in Chapter 5, including case studies in Appendix 5.1 for selected key risk sites. Information obtained specific to Whitecliffs and Te Kuha is incorporated into site models in Chapters 6 and 7 respectively.

## CHAPTER 4: LANDSLIDE SUSCEPTIBILITY

### 4.1 Introduction

The focus of Chapter 4 is landslide susceptibility and zonation within the project area. The primary objective of Chapter 4 is to present a qualitative landslide susceptibility zonation map based on the dataset developed in Chapter 3, and physical characteristics of the project area.

The two basic principles to consider in preparing a landslide susceptibility zonation map, as outlined by Fell et al (2008a), are: (1) ‘The past is the guide to the future’, in meaning that areas which have experienced landsliding in the past are likely to experience landsliding in the future; and (2) Areas with similar topography, geology and geomorphology as the areas which have experienced landsliding in the past are also likely to experience landsliding in the future. There are exceptions to these assumptions, including a landslide that exhausts source material after one event, but the general principles can be easily adopted for the project area.

The most basic method of describing a slope or area is as ‘susceptible’ or ‘not susceptible’ to landsliding. In addition, susceptibility mapping descriptors can be either qualitative or quantitative. Examples provided by Fell et al (2008a) for natural slopes are provided on Table 4.1 of both types of descriptors applicable to rockfalls, small and large landslides. The volume of material applied to the ‘small’ and ‘large’ landslides was not specified by Fell et al (2008a).

**Table 4.1: Landslide susceptibility mapping descriptors (based on Fell et al, 2008a)**

Susceptibility Descriptor		Rockfalls	Small landslides on natural slopes	Large landslides on natural slopes
Quantitative	Relative	Geomechanical descriptors (e.g. SMR)	Scores of contributing factors from data treatment techniques	<i>No detail provided by Fell et al (2008a)</i>
	Absolute	Factor of safety values from stability models	Factor of safety values from stability models	Factor of safety values from stability models
Qualitative	Field geomorphological analyses	Presence or absence of instability factors	Number of landslides per square kilometre	Presence or absence of landslides and their degree of preservation
		Density of scars on a rock slope	Percentage of area covered by landslide deposits	Presence or absence of activity indicators
	Index or parameter map	Overlapping of index maps with or without weighting		<i>No detail provided by Fell et al (2008a)</i>

There are 60 reported landslides in the Lower Buller Gorge project area listed in Table 3.5. The estimated volume(s) vary between ‘very small’ ( $<10\text{m}^3$ ) and ‘very large’ ( $>10,000\text{m}^3$ ). To display the inventory data in an easily assessable format, the following table summarises the location and assigns a single volume estimate category for each entry between small ( $<100\text{m}^3$ ), medium (100 to  $1,000\text{m}^3$ ) and large ( $>1,000\text{m}^3$ ). A fourth category is required for landslides with unknown volumes because of the large proportion of occurrences (25%) that did not have a volume estimate reported. Volumes assigned are weighted towards the larger end of the scale in cases where a range is defined on Table 3.5.

**Table 4.2: Individual landslide locations (SNL km) and assigned volume category**

Metrage (SNL km)	Volume category	Metrage (SNL km)	Volume category	Metrage (SNL km)	Volume category
95.96	Medium	111.00	Small	118.80	Large
96.00	Medium	111.04	Unknown	118.86	Large
97.50	Small	111.57	Medium	119.00	Unknown
97.50	Unknown	111.63	Medium	119.00	Medium
97.74	Unknown	112.00	Small	119.00	Unknown
98.00	Small	112.92	Medium	119.00	Unknown
101.65	Small	112.96	Medium	120.00	Small
102.80	Unknown	113.00	Small	120.00	Medium
104.80	Small	113.00	Large	121.00	Large
104.85	Small	113.80	Large	121.50	Large
105.05	Small	114.00	Large	122.43	Medium
105.30	Unknown	115.95	Small	123.10	Unknown
105.60	Unknown	116.00	Unknown	123.80	Small
106.65	Large	116.00	Small	124.00	Medium
107.50	Large	116.50	Small	124.27	Large
107.50	Small	116.68	Unknown	124.40	Medium
107.55	Large	117.28	Unknown	125.00	Large
108.60	Medium	117.44	Unknown	125.40	Large
108.63	Large	117.48	Large	125.50	Small
110.95	Small	118.50	Small	125.80	Unknown

Of the 60 reported landslides in the project area to date, 15 are classified as large; 12 as medium; 18 as small; and 15 as unknown. The ‘small’ category incorporates incidents of single rocks or trees falling onto the rail corridor. Limitations of landslide inventories are considered the greatest source of error in landslide zoning (Cascini et al, 2005). The methodology and limitations in developing the present landslide inventory have been documented in Chapter 3.

It is recognised that the steep natural slopes present above the majority of the rail corridor between SNL96 and 126km are critical for determining landslide susceptibility, particularly in conjunction with high annual rainfall experienced in the region and seismic activity. The spatial distribution of landslide occurrences from the inventory developed in Chapter 3, and Table 4.2, are presented in the following sections in regards to landslide volume, slope angle and geology. Based on the figures presented, a qualitative landslide susceptibility zonation map is developed, and methodology described, in Section 4.5.

## 4.2 Spatial trends

Locations and volume estimates from the landslide inventory are shown on Figure 4.1 to illustrate the spatial distribution of known and/or reported landslide occurrences. Colours assigned to the landslide events reflect the magnitude (volume of material released). The volume scale used is as specified in the legend of Figure 4.1. To interpret the dataset, it is important to note the following:

- Metrage locations are not shown to avoid visual confusion with symbology associated with landslide occurrence.
- The scale of the symbology used is for comparison purposes and does not reflect the areal extent of the landslide.
- 53 of the 60 landslide events listed in Table 4.2 are clearly depicted on Figure 4.1. The seven occurrences not shown are due to the close proximity of a number of locations, including: one large event at 107.55km; medium events at 95.96 and 112.96km; one small event at 97.50km; one unknown volume events at 97.74km and two unknown volume events at 119km.

A complete inventory would typically show a comparatively larger number of  $<100\text{m}^3$  (very small to small) landslides in comparison to  $100\text{--}1000\text{m}^3$  (medium) or  $>1,000\text{m}^3$  (large) volume landslides. The absence of more ‘very small’ to ‘small’ landslides reflects the incomplete reporting of these comparatively lower magnitude, but higher frequency, events. Landslide volumes in this order, particularly very small volumes  $<10\text{m}^3$ , will not necessarily cause a major disruption to rail operations.

The density of landslide occurrences increases from east to west, with a higher frequency of larger magnitude events between Hawks Crag Breccia (from SNL110km) and the western end of the project area at SNL126km (labelled near Te Kuha on Figure 4.1). Historic failures of varying magnitudes have undoubtedly occurred at Whitecliffs prior to the rail construction in the 1940s. This also applies to the remainder of the project area due to the steep topography and natural processes that occur over geologic time. The purpose of Figure 4.1 is to characterise reported landslides based on the current inventory.

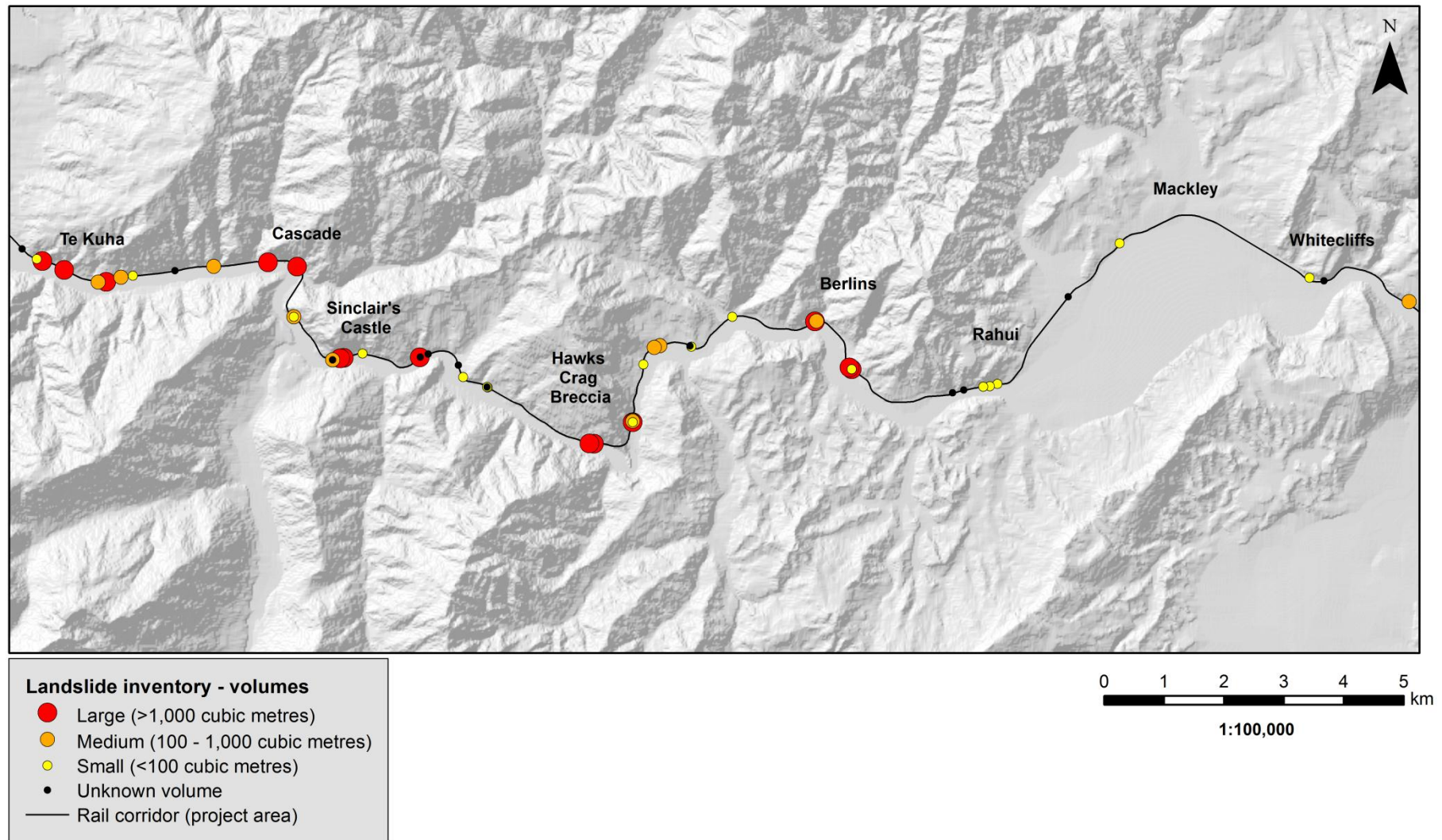


Figure 4.1: Spatial distribution of reported landslide occurrence based on volume estimates through the project area.

### 4.3 Slope analysis

Steep topography characterises the Lower Buller Gorge and, combined with high annual rainfall, is one of the main contributing factors to slope stability in the project area. Fell et al (2008a) and AGS (2007) suggest the following physical topographical characteristics in terms of determining landslide susceptibility that are relevant to the project area:

- Near vertical cliff-faces
- Natural slopes steeper than 35° (landslide travel is likely to be rapid)
- Natural slopes between 20° and 35° (rapid landslide travel is possible)
- Steep rail cuttings

Near-vertical cliffs within the project area are primarily limited to Whitecliffs, around SNL97.50km. Steep cuttings are also present in the gorge but the vertical and lateral extent is restricted due to the construction practice of attaching the rail to the natural contour of the Lower Buller Gorge hillslopes.

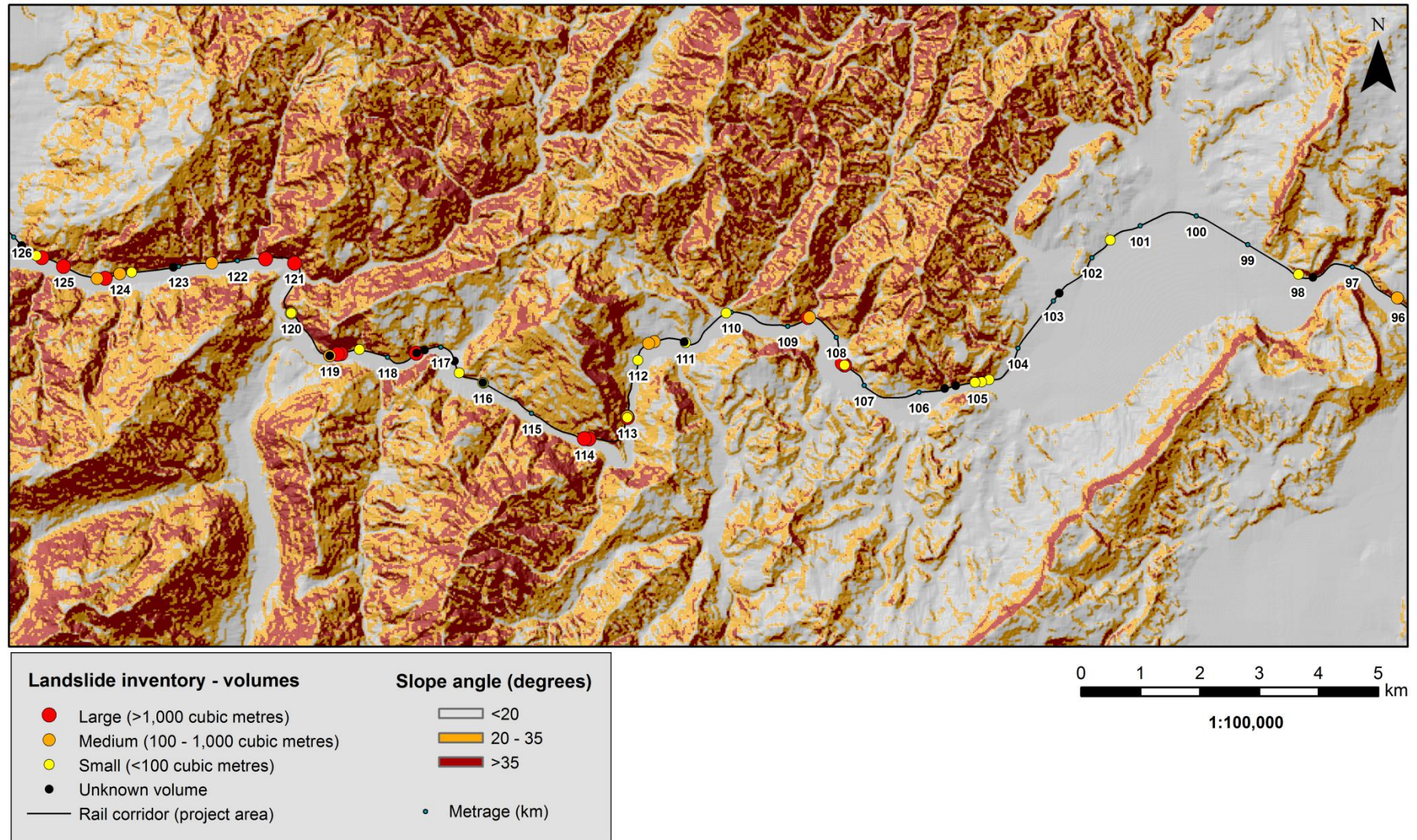
Based on the slope categories recommended by Fell et al (2008a), a digital elevation model (DEM) for the project area was used to represent slope angles of <20°, 20-35° and >35°. The cell size for the DEM shown in Figure 4.2 is 25m x 25m. The area classified on Figure 4.2 as >35° is considered to be representative of natural slope angles adjacent to the rail due to the absence of high cut slopes within the track formation.

Landslide occurrence is overlain on Figure 4.2, as per Figure 4.1, to show the relationship between topography and the available historic record of landslide events. The symbology size has been reduced to increase the visibility of slope angles adjacent to the rail corridor. Location references above the alignment have also been removed for the same purpose and any location references made in Section 4.3.1 can be correlated with Figure 4.1.

#### 4.3.1 Interpretation

With the exception of landslides from SNL110.0 to 112.0km, and SNL101.5 and 103.5km, the locations shown on Figure 4.2 typically correlate with slope angles greater than 35°. Considering the proportion of the rail corridor that has slope greater than 35° immediately adjacent to it makes this correlation not surprising and landslide susceptibility is considered comparatively high for the majority of the project area based on topography alone.





**Figure 4.2: Spatial distribution of reported landslide occurrence in relation to slope angles above the rail corridor**

Other main features of Figure 4.2 are summarised below (discussed from east to west through the project area):

- Small to medium volume landslides associated with the low elevation (around 140m amsl) hillside in the vicinity of SNL96km correlate to a narrow band of slope angles  $>35^\circ$ .
- Landslide occurrence at Whitecliffs will always be associated with slope angles  $>35^\circ$  due to the near vertical cliff-face. No medium or large-scale rockfalls ( $>1,00\text{m}^3$ ) have been reported in the project area since the rail became operational in the 1940s.
- Only two landslides are shown in the 6km length of rail corridor between SNL99 to 104km. This length of rail is associated with slope angles predominantly  $<20^\circ$ .
- Variable slope angles are associated with hillsides (typically less than 450m amsl) between SNL104 and 110km, which accounts for a range of landslide occurrences. There are ten shown in this 6km section of rail, including two large landslides at SNL107.50km and one large volume event at SNL108.63km (partially obscured by a medium volume landslide at SNL108.60km).
- Six landslides of small, medium and unknown volumes are shown between SNL110 and 112km associated with slopes predominantly less than  $<20^\circ$ .
- The terrain in the vicinity of Hawks Crag Breccia (SNL113km) is dominated by slope angles  $>35^\circ$  on the east facing hillside and a range of landslides are shown of all volume categories.
- Two large landslides recorded on the western side of Hawks Crag Breccia are confined within in a relatively small area of steeper terrain  $>35^\circ$  (near the western portal of Tunnel 3). The highest elevation point above this section (Mt Cassin) is 706m amsl.
- Between SNL117 and 126km the topography immediately adjacent to the rail alignment is dominated by the  $>35^\circ$  slope angle class. This 9km length of rail corridor also accounts for eight of the recorded large to very large landslides (out of fifteen for the entire 30km project area length, as listed in Table 4.2). The maximum elevation of the mountain ranges north of the rail exceeds 800m amsl (north to northeast of Cascade).

Geological controls on slope stability are an influencing factor, particularly for areas of the rail corridor that have an absence of reported large volume landslides associated with slopes  $>35^\circ$ , which includes Whitecliffs. Consideration of geological influence on slope stability is presented in the following section.

#### 4.4 Geological controls

A geological map developed from data supplied by GNS Science (QMAP raster data based on Nathan et al, 2002) is presented in Figure 4.3 with reported landslide occurrence overlain to determine any discernible correlations between slope failure and geology. Metrage locations are provided below the geological map for reference at the same map scale.

The three main faults are labelled on Figure 4.3 (Lower Buller, Mt William and Inangahua). The Mount William Fault trace terminates at the Buller River, but it is assumed to continue south of the project area. Fault locations shown are consistent with available QMAP spatial data provided by GNS Science. Fault-bounded contacts are also shown between granite and Greenland Group outcrops near SNL118.50km (Ohika Fault, inactive), and between granite and Kaiata Formation around SNL103.50km.

#### 4.4.1 Interpretation

The Buller River cuts through the Paparoa Anticline, which extends from south of Greymouth to the Buller River, and is a major developing tectonic feature (Tulloch and Kimbrough, 1989). The progressive incision of the Buller River coupled with the high annual rainfall, and periodic stripping of surficial materials, maintains a shallow regolith unit on most of the slopes above the rail corridor.

The distribution of landslides is not confined to a particular geological unit (Figure 4.3), but there is an increase in landslide density towards the west. This infers that weathered granite and Hawks Crag Breccia (from around SNL110km to 126km) have a comparatively higher susceptibility for landslide generation, which is also related to the steep slopes in the area, and to defect (joint) controlled failures. A summary table is provided in Table 4.3 which divides the rail corridor into seven categories based on the predominant geological unit. The length and number of landslides reported within each category is also shown on Table 4.3.

Detail in Table 4.3 illustrates that landslide occurrence is greatest in Hawks Crag Breccia and the granitic rocks from SNL110km to the western extent of the project area at SNL126km. This 16km length of rail accounts for 68% of the total (60) reported landslide occurrences. The absence of landslides within the short lengths of rail corridor from SNL109.0-110.0km (Berlins Porphyry) and SNL118.0-118.5 (Greenland Group) does not decrease the susceptibility of these two locations to future landslide events, particularly at SNL118.0-118.5km where the slope angle is  $>35^\circ$  immediately adjacent to the track formation (Figure 4.2).

The absence of large volume rockfalls or topples originating within limestone outcrops at Whitecliffs on the northern side of the Buller River is in stark contrast if the southern outcrop of limestone adjacent to SH6 is considered. The earthquake-generated failures in 1968 that resulted in a fatality and impacts to road infrastructure from Whitecliffs on the southern side of the Buller River are controlled by bedding and joint orientation. These structural controls for stability are further discussed in Chapter 6 but essentially the limestone on the north bank dips at gentle angles ( $<10^\circ$ ) into the face and joints are very widely spaced ( $>10\text{m}$ ).



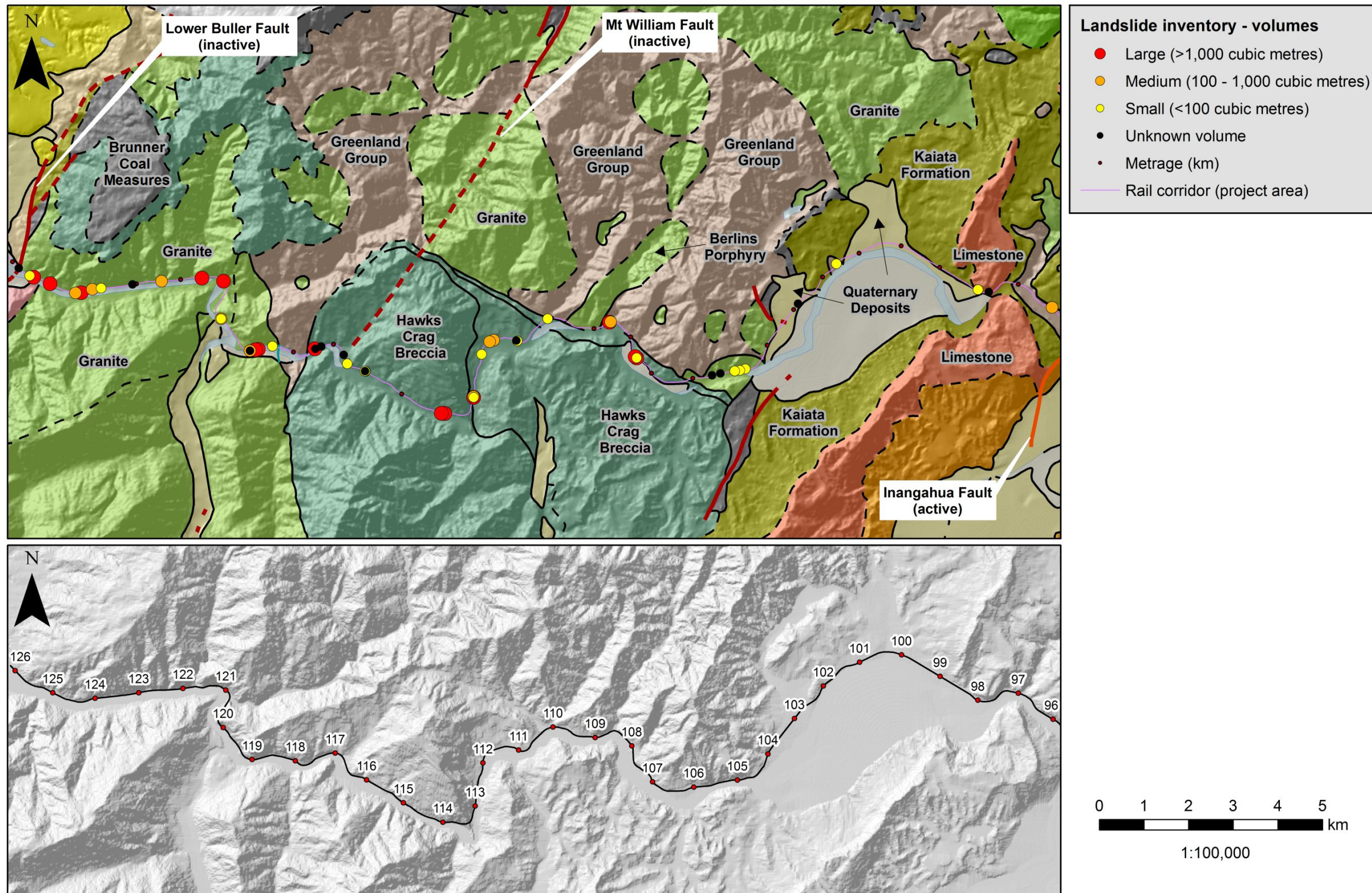


Figure 43: Spatial distribution of reported landslide occurrence and main geological units within the project area (based on GNS Science QMAP data and Nathan et al, 2002). Reference metrage locations are shown in the lower section at the same map scale.

**Table 4.3: Landslide occurrence and geological units**

SNL metrage (km)	Predominant geological unit	% of rail corridor <sup>(A)</sup>	% of landslides <sup>(B)</sup>	Volume range <sup>(C)</sup>
96.0 - 97.0	Kaiata Formation	3% (1.0km)	3.5% (2)	Medium
97.0 - 99.0	Limestone (Whitecliffs)	7% (2.0km)	7.0% (4)	Small and unknown
99.0 - 103.5	Quaternary deposits	15% (4.5km)	3.5 % (2)	Small and unknown
103.5 - 109.0	Various <sup>(D)</sup>	18% (5.5km)	18% (11)	Small to large
109.0 - 110.0	Berlins Porphyry	3% (1.0km)	0	-
110.0 - 118.0	Hawks Crag Breccia	27% (8.0km)	33% (20)	Small to large
118.0 - 118.5	Greenland Group	2% (0.5km)	0	-
118.5 - 126.0	Granite <sup>(E)</sup>	25% (7.5km)	35% (21)	Small to large
Notes: (A) Approximate extent of outcrop based on available mapping expressed as a percentage, and length in brackets (B) Percentage and number of landslides (in brackets) recorded within specific geological units (refer inventory in Table 4.2, total of 60 landslides) (C) Volume range based on Figure 4.3 and inventory details from Table 4.2 (D) Includes Kaiata Formation around 107.50km (E) Schistose Greenland Group inclusions noted in vicinity of Te Kuha				

The extent of deep weathering within bedrock through the Lower Buller Gorge is minimal due to high annual rainfall, and subsequent periodic removal of surficial materials as a function of normal weathering and erosion processes. There is limited potential for development of regolith materials under these conditions, and typical depths in landslide headscarps are in the order of 2-3m, based on observations along the rail corridor during site inspections between 2009 and 2011. This is confirmed by relatively high intact rock strengths obtained using Schmidt Hammer testing at Te Kuha (granite and Greenland Group) where rock sampling and analysis was undertaken. Results and detailed discussion regarding these tests are provided in Chapter 7.

## 4.5 Landslide susceptibility zonation map

A qualitative landslide susceptibility zonation map is provided on Figure 4.4 showing a relative scale of 'high', 'moderate' and 'low' for the Lower Buller Gorge project area. The rationale for the relative scale is based on the susceptibility mapping presented in this chapter, as outlined below:

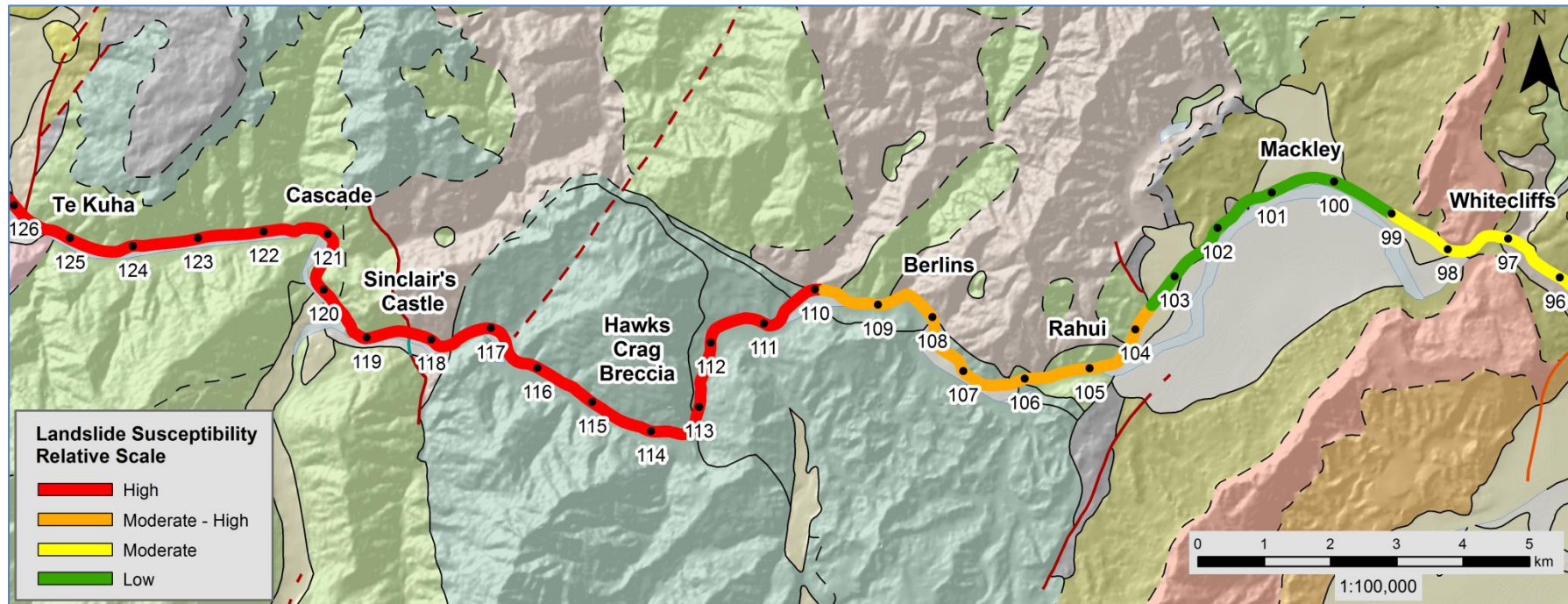


- **‘High’ susceptibility:** incorporates SNL110.0 to 126.0km, which accounts for the steepest slope angles (typically  $\geq 35^\circ$ ), and 68% of all landslides reported for the project area to date, including 12 of the 15 ‘large to very large’ volume landslide occurrences. Hawks Crag Breccia and granitic rocks are the dominant geological units between SNL110km and 126km. Steep catchments above this length of rail are comparatively higher (up to ~800m amsl above Cascade) than the remainder of the rail corridor, which results in a higher volume of source material availability for landslides.
- **‘Moderate to High’ susceptibility:** shown on Figure 4.4 between SNL103.5 and 110.0km. The remaining three ‘large to very large’ volume landslides (of 15) occurred within this length of rail. The landslide inventory indicates 18% of the landslide occurrences take place in this section, in Kaiata Formation and Greenland Group bedrock. Slope angles are predominantly  $\geq 20^\circ$ , with some areas also  $\geq 35^\circ$ . Adjacent hillsides are generally around a maximum of 450m amsl, allowing for comparatively less source material availability than the ‘High’ category described above.
- **‘Moderate’ susceptibility:** shown on Figure 4.4 between SNL96.0 and 99.0km, including Whitecliffs limestone outcrops. No large volume landslides have been reported in this section of the rail corridor since the track became operational in the 1940s. The near vertical cliffs around SNL97.5km have released many limestone blocks of varying sizes over time, but most are captured within the well-established talus apron and only four small or unknown volume incidents have been recorded around Whitecliffs to date. Shallow regolith and/or colluvium failures around SNL96km account for <5% of the total landslide occurrences, but other failures of small to medium volumes are likely in the future based on detailed API and site inspections.
- **‘Low’ susceptibility:** applied to the section of rail between SNL99.0 and 103.5km due to the slopes immediately adjacent to the rail corridor being predominantly  $< 20^\circ$ , mainly within Quaternary deposits, and only two recorded landslides have been reported in this 4.5km length of rail to date (<5% of the total).

Small sections of rail corridor within ‘High’ and ‘Moderate - High’ zones will have lower susceptibility, but have not been separated at the map scale presented in Figure 4.4. The present map is based on desktop information only, and can be refined by site-specific investigation as at Te Kuha between SNL124 and 126km (Chapter 7).

Susceptibility mapping can also take triggering mechanisms into account. All slopes that have a developed regolith or colluvium layer (typically  $\leq 3$ m depth within the project area) are susceptible to rainfall-induced landsliding. In terms of earthquake triggering events, Wyllie and Mah (2004) lists five main parameters that they consider to have the most influence in relation to rock slope stability during seismic activity. These are outlined, together with a decision tree developed by Keefer (1992) that adopts the five parameters, and provides a ranking between ‘Extremely High’ and ‘Low’ in terms of his susceptibility of rock slopes to earthquake-generated failure (Figure 4.5).





**Figure 4.4: Qualitative landslide susceptibility zonation map. Metrage locations (SNL km) and main location references are overlain.**

Parameters listed by Wyllie and Mah (2004) that have the most influence in relation to rock slope stability during seismic activity are:

1. **Slope angle:** An angle of 25° or greater is typically required for rockfalls or slides to occur.
2. **Weathering:** Highly weathered rocks that comprise core material in a fine soil matrix of residual soil are most prone for failure.
3. **Induration:** Weakly bonded rock is more likely to fail.
4. **Discontinuity (defect) characteristics:** Closely spaced and open discontinuities within a rock outcrop are more at risk of failing than massive outcrops with closed discontinuities.
5. **Water:** Areas with high water tables or antecedent rainfall are most susceptible to failure.

Application of the decision tree process to rock outcrops at Whitecliffs and Te Kuha both result in a 'High' rank for susceptibility primarily as a result of climatic conditions and associated impacts of antecedent rainfall. Rock outcrops are often free draining, which could result in a 'Moderate' rank if the influence of rainfall is considered minimal for Whitecliffs. This type of screening tool is useful for broadly classifying slope stability hazards.

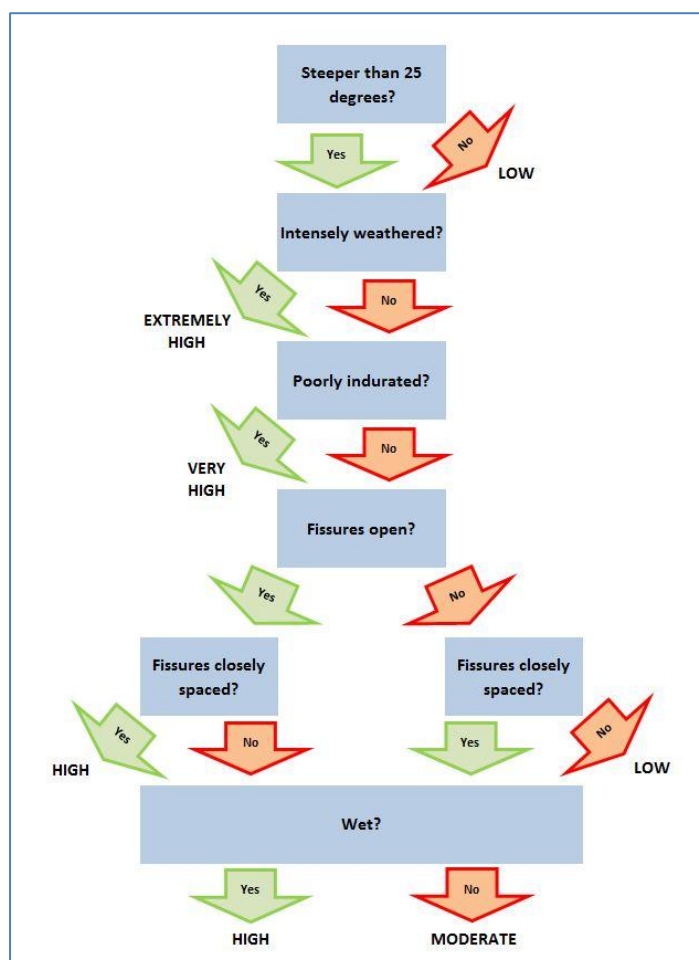


Figure 4.5: Decision tree for earthquake-generated landslide susceptibility of rock slopes (based on Keefer, 1992).

## 4.6 Summary

Chapter 4 has focused on developing a qualitative landslide susceptibility map and zonation of the 30km rail corridor through the Lower Buller Gorge. The zonation map was intentionally designed for a simplistic overview of the project area in terms of landslide susceptibility based on a desktop evaluation of key controls for slope stability (topography and geology) correlated with actual landslides occurrences.

Four zones have been identified in which the landslide susceptibility has been defined as ‘high’, ‘high to moderate’, ‘moderate’ or ‘low’. The key conclusion from the susceptibility mapping and analysis is that ~75% by length of the rail corridor through the Lower Buller Gorge is classified as moderate to highly susceptible to landsliding from upslope sources. Susceptibility is a function of the high annual rainfall (typically between 2,000 and 4,000mm per annum), long steep ( $\geq 20^\circ$ ) slopes, and the absence of catch or detention areas above track level. Chapter 5 follows on from assessing landslide susceptibility to landslide hazards, including a detailed overview of the primary triggering mechanisms for slope instability affecting rail operations from upslope sources.

## CHAPTER 5: LANDSLIDE HAZARD

### 5.1 Introduction

‘Landslide Hazard’ in risk management terminology considers the annual probability of occurrence based on both actual and potential landslides (susceptibility) for a specified area. The landslide inventory established in Chapter 3 presents recorded incidents of slope instability only, but does provide a basis for analysis. Limitations for determining annual probability based on data collected to date for the Lower Buller Gorge project area include:

- Of the 60 landslides in Chapter 3 (Table 3.5) only half of these (30) had a specific date associated with the event due to incomplete documentation of impacts to the rail corridor.
- Records made available by KiwiRail are applicable from 2004, which accounts for 60% (18) of the 30 events with a specific date.
- Nine of the events (30%) with dates are associated with the 1968 Inangahua earthquake, based on research conducted to date.
- The frequency of reported landslide occurrences that have impacted rail operations since 2004 is around 3 to 4 per year, which is considered an underestimation of the actual number.
- Landslide frequency would vary in terms of magnitude; frequency of high intensity, or long duration, rainfall triggers; and episodic seismic activity.

In determining the feasibility of developing a quantitative hazard or risk zonation map, Fell et al (2008a) states that: ‘*Quantitative hazard and risk zoning cannot be performed where data on frequency of landslides either do not exist or are so uncertain as not to be relied on*’ (Fell et al, 2008a, page 89). In the Lower Buller Gorge rail corridor detailed temporal analysis is not realistic given the limited data-set for the project area but despite the limitations, ‘Landslide Hazards’ and triggering mechanisms can be still be characterised. The objectives of Chapter 5 are therefore to:

- Determine the spatial distribution of landslides based on triggering mechanism (rainfall-induced and earthquake-generated) and frequency of occurrence.
- Provide a temporal case study of rainfall-induced landslides that occurred within the project area in late December 2010 and early January 2011.

In addition, detailed summaries of three key risk sites identified during discussion with KiwiRail personnel (Chapter 3, Section 3.4.4) that have exhibited medium to large volume landslides ( $100\text{m}^3$  to  $>1,000\text{m}^3$ ) or high frequency (a one incident every two years) are presented Appendix 5.1. These three locations are at SNL107.5, 118.5 and 121.0km.

## 5.2 Triggering mechanisms

Earthquake-generated and rainfall-induced landslides are recognised as the predominant triggering mechanisms for slope failures in the Lower Buller Gorge project area. The spatial distribution for each category, including those with unknown triggering mechanisms is shown in Figure 5.1, based on data from the landslide inventory presented in Chapter 3 (Table 3.5). The percentage of each category identified in the project area is also presented graphically in Figure 5.1.

Fifty-five percent of entries in the landslide inventory (Chapter 3, Table 3.5) are directly attributable to rainfall as the triggering mechanism. The high proportion of entries having an unknown triggering mechanism (27%) reflects incomplete documentation, and rainfall is considered more realistically accountable for up to approximately 80% of all entries. Eighteen percent of landslides are listed on Table 3.5 as being generated by the 1968 Inangahua Earthquake.

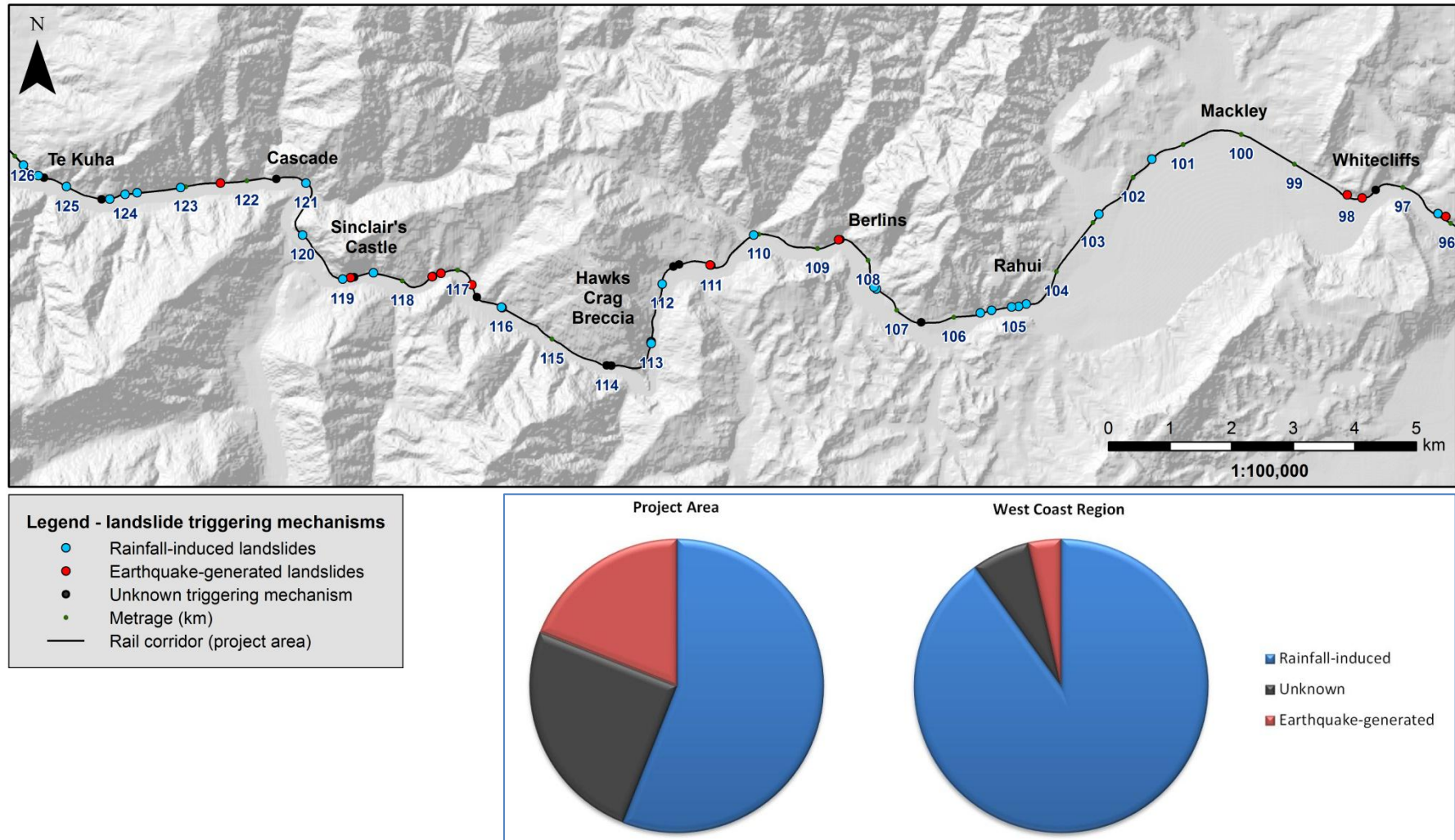
In comparison to the data collated specific to the project area, the proportion of triggering mechanisms attributable to landslide generation for the entire West Coast region is also shown in Figure 5.1, based on research conducted by DTec Consulting Ltd (2002). The different relative percentages between the two pie graphs in Figure 5.1 is the higher weighting of site specific data from the Lower Buller Gorge project area to landslides generated by the Inangahua Earthquake. The DTec Consulting Ltd (2002) inventory (Appendix 3.1) incorporates the triggering mechanism as a single event, and not the resultant number of landslides generated.

## 5.3 Rainfall-induced landslides

Rainfall is undoubtedly the most dominant triggering mechanism for slope movement in the project area in terms of landslide frequency. The annual rainfall experienced in the Lower Buller Gorge is typically greater than 2000mm, at times double this figure. Rainfall intensity can vary between adjacent tributary catchments through the Lower Buller Gorge during rainfall events. The establishment of rainfall triggering levels is recognised as an important component for Landslide Risk Management within the project area.

As outlined in this section, there are considerable gaps in the regional rainfall gauge network that makes correlation to actual landslide occurrence difficult to accurately assess. A temporal case study is presented in Sections 5.4 for a rainfall event classified by WCRC as 1 in 10-year. Observed impacts to the rail corridor are outlined in Section 5.5. For comparison, a case study by Jaiswal and Weston (2009) is presented within Section 5.3.3 to highlight recent research in this area applicable to a rail corridor, and emphasising the importance of measurement of local catchment rainfall.





**Figure 5.1: Spatial distribution of reported landslide occurrence and triggering mechanism, including pie graphs showing the proportional representation of available data for the project area (this study) and West Coast Region (based on DTec Consulting Ltd, 2002)**



To enable the frequency of rainfall-induced landslides to be established from the historic record, rainfall intensity and duration data specific to the Lower Buller Gorge requires analysis. Using the rainfall data for actual recorded landslide occurrences, and establishing the return period of the event, would provide an indication of long-term frequency. Discussions with staff from WCRC indicate an absence of rainfall recorders within the project area, which is recognised as a gap in the network.

The closest two rainfall recorders within the WCRC network are located at Westport and Reefton. Rainfall also appears to be recorded at Landing (10km upstream of the Inangahua and Buller River confluence). Data from this site was summarised in Chapter 2, Table 2.1. While rainfall data from these locations could be used to correlate landslide occurrences, the spatial variability is considered too high to allow for an accurate assessment for rainfall-induced landslide triggering levels, particularly for lower magnitude events that generate very small to small volume ( $<100\text{m}^3$ ) landslides. The consideration of antecedent rainfall also needs to be incorporated into any model for developing threshold levels, as discussed in Section 5.3.2.

In the absence of site specific rainfall data, a literature review was conducted to determine the availability of relevant research regarding rainfall as the predominant triggering factor for slope stability issues, including antecedent patterns and case studies. Bell (1976; 1994) established the importance of 6-hour and 12-hour rainfall durations in initiating catchment-wide slope stability, including debris slides and flows affecting the South Island Main North Line, during Cyclone Alison in March 1975. The following sections summarise the findings of the literature review, including New Zealand and international research from the following sources: Rahardjo et al (2001); Borja and White (2010); Jaiswal and Weston (2009); and Glade (1998).

### 5.3.1 New Zealand literature

Previous studies related to the determination of reliable thresholds for triggering rainfall-induced landslides, dated from the 1980s, was summarised by Glade et al (2000) as all requiring a robust database of historic occurrences. In New Zealand there are a number of databases available, including landslides of a specific magnitude (IGNS, 1993), and rainfall data that triggered ‘highly damaging’ landslides (Harmsworth and Page, 1991). This inventory factor should be recognised for any hazard assessment conducted in the area. Future reporting of incidents, in an appropriate manner, is necessary to enable the development of more reliable rainfall thresholds in the future.

The frequency and magnitude of rainfall-induced slope movement is a relevant field of research for all areas of New Zealand. Relevant papers on the topic specific to New Zealand are dominated in the late 1990s to 2000 by Thomas Glade, including: Glade et al (2000); Glade and Crozier (1997); Glade (1997 and 1998).

The basis of research for the Glade et al (2000) paper was that applying data from the climatic record in New Zealand makes it possible to differentiate between input conditions that did, or did not, trigger landslides. Rainfall thresholds can subsequently be developed for specific regions. Glade et al (2000) also discuss that temporal variability in climatic regimes does not affect the rainfall threshold, but the frequency that the threshold is exceeded. Importantly for the project area, this also implies that for an area with a high annual rainfall that could potentially trigger regolith or rock slope failures, the likelihood of future occurrences for a particular location can at times be reduced due to decreased availability of material on the slope. This could be applied to the shallow failures observed around SNL95-97km in February 2011, where the likelihood of future occurrences of a similar nature is limited due to the exposure of bedrock (refer Section 5.5). Instead, the mechanism of failure is applicable to comparable slopes that have not failed, or not reported to have failed, during the operational history of the rail corridor.

Research by Glade (1998) indicated that the project area is in a region that experiences a comparatively higher frequency of landslide-triggering rainfall events, when considered on a national scale. Glade (1998) classified the region as experiencing one storm every 2-4 years that results in a landslide, based on recorded landslide triggering rainfall in New Zealand between 1870 and 1995. It is assumed that the magnitude of the landslide generated is of sufficient volume to cause disruption and/or inconvenience to impacted areas, otherwise the frequency for smaller-scale events would be a lot higher. It is accepted that the definition of landslide impacts is subjective.

Glade (1998) also conducted regional scale analyses for three areas (Hawke's Bay, Wairarapa and Wellington) and correlated landslide-triggering rainfall events with 24-hour precipitation data. All areas resulted in a minimum probability threshold of 20mm daily rainfall to trigger a landslide. The maximum probability threshold varied between 120mm/24-hours and 300mm/24-hours. The 24-hour data does not take antecedent conditions into account and Glade (1998) recognised this as a topic for future research. The significance of antecedent moisture conditions is relation of landslide triggering is well recognised in New Zealand (Bell, 1976; Eyles et al, 1978).

### **5.3.2 Antecedent rainfall**

Rahardjo et al (2001) highlights the role of antecedent rainfall on slope stability through a case study based in Singapore. Rainfall intensity measuring 95mm in 150 minutes was record in February 1995 in Singapore that caused more than 20 shallow landslides. Similar magnitude events presented by Rahardjo et al (2001) did not trigger landslides and the cause for the February 1995 slope movements was interpreted as being due to antecedent rainfall patterns during the preceding five days. This conclusion was supported by numerical modelling for a specific slope that simulated different rainfall patterns, and calculated differences in the factor of safety for the slope.

The quantification of hydrologic and geotechnical processes that govern slope failures are considered by Borja and White (2010) to remain inadequate, despite extensive research in this area and development of many slope-stability models. The simplification of physics, and neglect of partial saturation effects, is the main limitation identified within existing models. Borja and White (2010) describe the influence of rainfall infiltration on a slope with the following main points:

- The surface tension between soil particles will eventually break when saturation levels are increased during periods of intense or prolonged rainfall;
- A down-gradient frictional drag on the slope will occur when the inter-granular fluid flow within a soil becomes mobilised; and
- When the capacity for rainfall to infiltrate slope soil is exceeded, surface runoff and associated erosion processes will result.

The case study presented by Borja and White (2010) adopted a physics-based approach for quantifying rainfall-induced slope deformation and a finite element model that considers both solid deformation and fluid pressure in an unsaturated soil. The main finding of the research highlights the limitations of modelling complex processes and cautions against applying simplified analyses for hazard mitigation purposes.

### 5.3.3 Case study: Southern India

A case study by Jaiswal and Weston (2009) was the most relevant research article obtained in regards to a rail corridor project. The paper focussed on the establishment of rainfall thresholds using a temporal probability model for cut slopes adjacent to a 19km length of rail in southern India. The model developed by the authors provides the likelihood of occurrence of rainfall that can trigger landslides with a particular density per unit area.

The study area was chosen to test the methodology adopted based on the availability of a complete landslide inventory between 1987 and 2007, including date of occurrence. A total of 790 landslides were recorded within the study area (25km<sup>2</sup>). Information was predominantly sourced from rail maintenance records, and other technical reports. In addition to a robust set of landslide data, there are 15 rainfall gauges within the study area. The main difference between the rail corridor studied by Jaiswal and Weston (2009) and the Lower Buller Gorge project area is the proportion of landslides originating in cut slopes. In the Southern India case study, cut slopes accounted for 94% of events (which were defined as failures due to rainfall). This reflects the rail construction methods and local geological characteristics.

The average landslide volume provided by Jaiswal and Weston (2009) was around 400m<sup>3</sup>, with a median volume of 50m<sup>3</sup>. These volumes are comparable to the Lower Buller Gorge project area and

highlight that despite the typically medium to small landslide events ( $\leq 500\text{m}^3$ ), the frequency (35-40 events per year in Southern India case study) causes considerable impacts to on-going rail operations. Jaiswal and Weston (2009) indicated that the rail company in Southern India spends around US\$250,000 on restoration works each year as a direct result of landslide damage. A summary of methodology, and main outcomes, of the research completed by Jaiswal and Weston (2009), are discussed below:

- The 19km rail corridor length was divided into sections based on topography.
- Rainfall thresholds were developed for the initiation of shallow translational debris slides and debris flows based on the database of landslide occurrences (790 incidents in total), and daily rainfall data for the same period that was obtained from a network of fifteen rain gauges.
- Rainfall data was used to establish the relationship between antecedent and daily rainfall patterns and landslide occurrence.
- The temporal probability calculation was a joint condition that considered both the annual exceedence probability of the rainfall threshold and the landslide occurrence probability once the rainfall threshold is exceeded.
- Rainfall events that could trigger one or more landslides within each section of the rail corridor required the definition of four thresholds.
- For the entire 19km length of rail, one rainfall threshold was developed that could potentially trigger 15 or more landslides.
- A range between 0.27 and 0.49 for the annual temporal probability was calculated.

The study area in Southern India was chosen on the basis of the availability of complete data sets for both rainfall and landslide occurrence. The model developed by Jaiswal and Weston (2009) still had many limitations despite the robust data sets. Model limitations included the non-applicability to other mechanisms of slope failure, and processes such as rockfall, that occur in the unmodified slopes above the rail corridor.

Applying a similar methodology to the Lower Buller Gorge project area is not recommended based on the limited rainfall data and lack of detailed incident reporting at present in regards to landslides. The Jaiswal and Weston (2009) model simplification of not considering natural slopes above the corridor would also need modification if a similar type of methodology was adopted for the Lower Buller Gorge project area where the source of slope movements that could impact rail infrastructure is not confined to the very limited extent of cut slopes.

Following on from the 2009 case study, Jaiswal et al (2010) used the same study area to apply a quantitative approach for landslide risk assessment applicable to transportation corridors. The method used enabled the estimation of direct (people, infrastructure and vehicles) and indirect risks

(economic). The methodology and approach is outside the scope of this current thesis, but does illustrate the application of landslide risk management as a facilitation tool for future mitigation and work prioritisation.

#### 5.4 Temporal case study (December 2010 to January 2011)

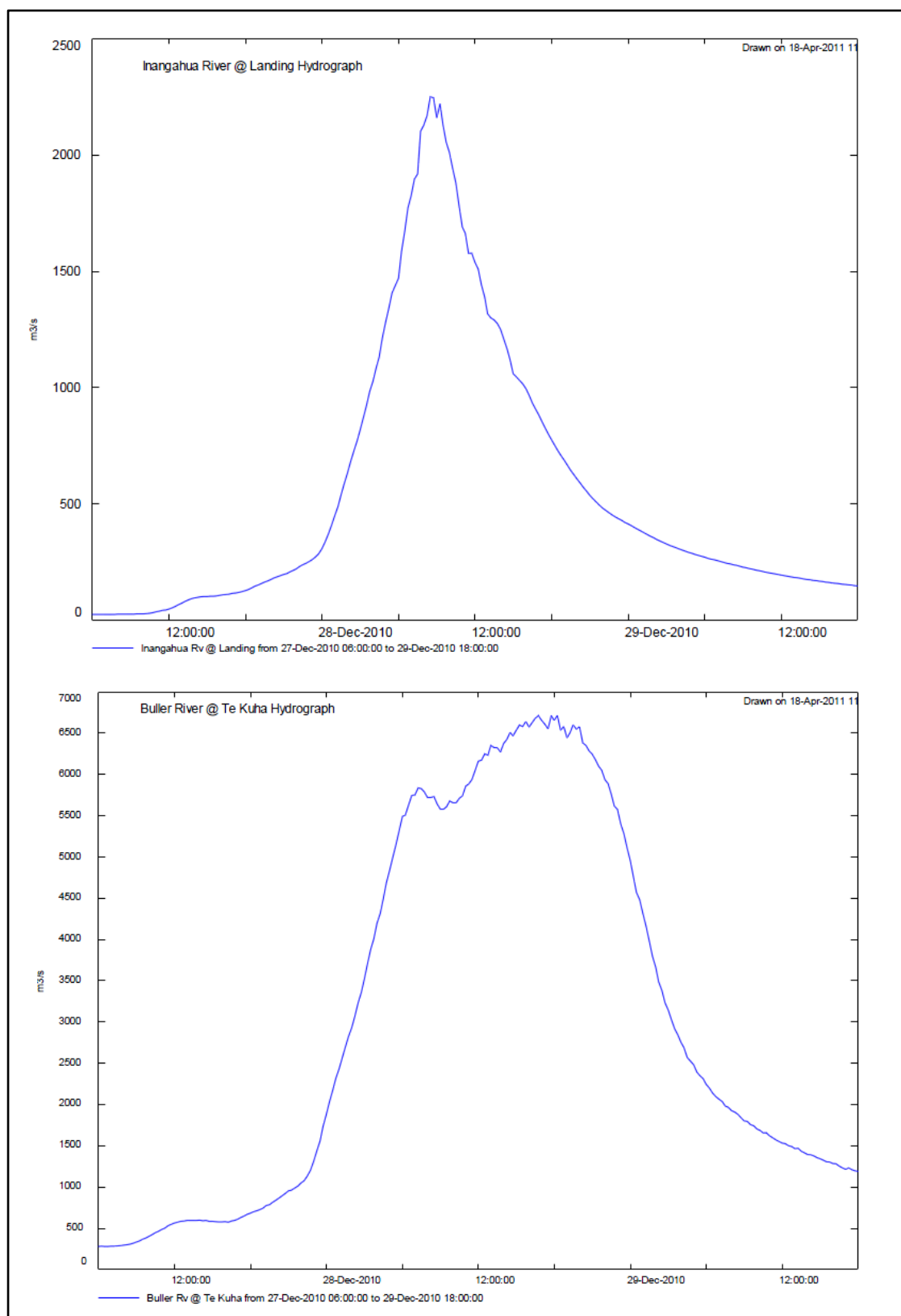
In December 2010 the West Coast Region experienced rainfall that caused flooding and landslides from Haast to Karamea. A report produced by the WCRC collated all available information to describe the extent of the flood event (WCRC, 2011). The publication date was not specified in the report but it is assumed to be early 2011. Data relevant to the project area is summarised in this section based on WCRC (2011) to highlight the impacts from rainfall-induced landslides, including a photographic log and descriptions of landslides observed during a site inspection as part of this thesis in February 2011 (Section 5.5).

The rainfall originated from a warm, moist north-easterly flow that was followed by a large cold front. Antecedent conditions were a contributing factor as the area was already moderately saturated (WCRC, 2011) by rainfall on 21 and 24 December 2010. Peak flow rates, and corresponding water levels; estimation of return periods; rainfall frequency analysis, and total daily rainfall between 21 and 29 December 2010, were reported by WCRC (2011).

Of the 16 water level monitoring sites listed in the West Coast Region, data from the three closest to the project area is provided in Table 5.1. Monitoring sites/recorder names are the same as referred to in Chapter 2, Section 2.2 of this thesis. Hydrographs from WCRC (2011) for the Inangahua River at Landing and Buller River at Te Kuha are shown on Figure 5.2. The hydrographs show the peak at Inangahua dropped comparatively more quickly than for the Buller River, which reflects the different catchment sizes.

**Table 5.1: Peak flow, water levels and return period analysis (WCRC, 2011)**

River and gauge reference	Date and time of peak flow	Peak level (mm)	Peak flow (m <sup>3</sup> /s)	Estimated return period (years)
Buller River (Te Kuha)	28/12/2010 at 1815	10,927	6,714	10
Buller River (Woolfs)	28/12/2010 at 1540	7,229	3,847	9
Inangahua River (Landing)	28/12/2010 at 0830	6,255	2,250	16

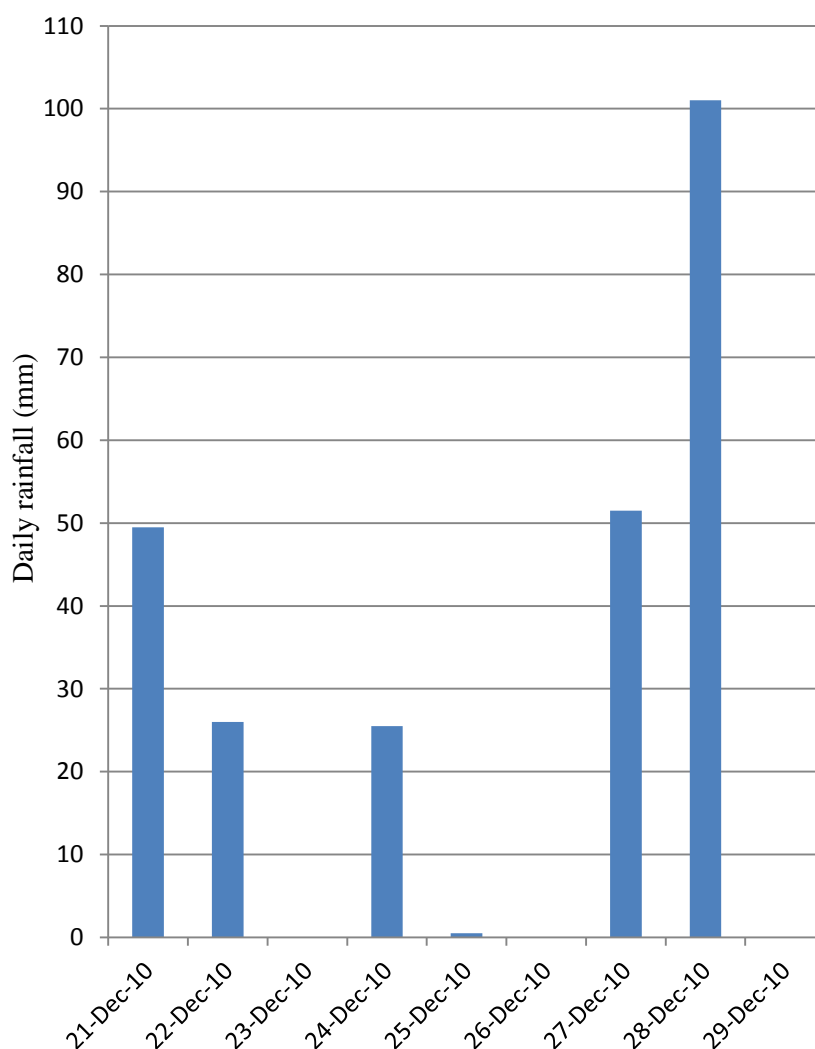


**Figure 5.2: Hydrographs from the 28 and 29 December 2010 for the Inangahua River at Landing (top) and Buller River at Te Kuha (bottom). Graphs sourced from WCRC (2011).**



Rainfall duration and frequency data is only available for the Inangahua River location at Landing. Information from WCRC for the period 26 to 29 December 2010 indicated that the rainfall intensity was greatest over a 12-hour period between 27 and 28 December 2010, with 117mm recorded, which correlated to a 10-year return period. Data from the West Coast Region as a whole lead WCRC to interpret the event as *'an intense medium to long term duration event with 6-24 hours producing significant rainfall depths and in some cases 48 hour significant falls'* (WCRC, 2011, p5).

The effect of antecedent conditions was recognised by WCRC, with daily rainfall measurements recorded for the Inangahua River Landing between 21 and 29 December 2010 shown on Figure 5.3. Rainfall in the region on 21, 22 and 24 December would have influenced the level of pre-existing soil saturation levels prior to the comparatively more intense and longer duration rainfall on 27 and 28 December 2010.



**Figure 5.3: Antecedent rainfall pattern from the rainfall gauge at Inangahua (Landing) between 21 and 26 December 2010 prior to the rainfall that caused landslides in the Lower Buller Gorge on 27 and 28 December 2010. Rainfall data obtained from WCRC (2011).**

Rainfall-induced landslides were not a focus of the WCRC (2011) report, but road closures were mentioned, including a closure for 1.5 days around Inangahua due to landslides. The Buller River flow levels and rates were considered moderate only (Table 5.1), but it was noted that landslides and road closures were frequent during the event throughout the Buller River catchment area. No direct reference to the rail corridor was made. Flooding also occurred in the region less than one month later (around 18 January 2011), which caused additional landslides and disruption to transportation routes, including the Lower Buller Gorge rail corridor. No report was identified related to the January 2011 rainfall event.

## **5.5 Impacts to the rail corridor (December 2010 – January 2011)**

The following photographic record and descriptions of failures types, material and volume estimates are provided for areas that were visited during a site inspection on 24-25 February 2011. Access was restricted due to the 22 February 2011 magnitude 6.3 earthquake in Christchurch that required rail personnel scheduled to provide assistance to be called to more urgent works in the response phase of the disaster. The following discussion is provided to highlight the types and small to medium volumes of landslides that impacted the rail corridor during rainfall events in December 2010 and January 2011, this being considered typical of the scale of landsliding causing track closure with a frequency of once per decade.

### **5.5.1 SNL95 to 97km**

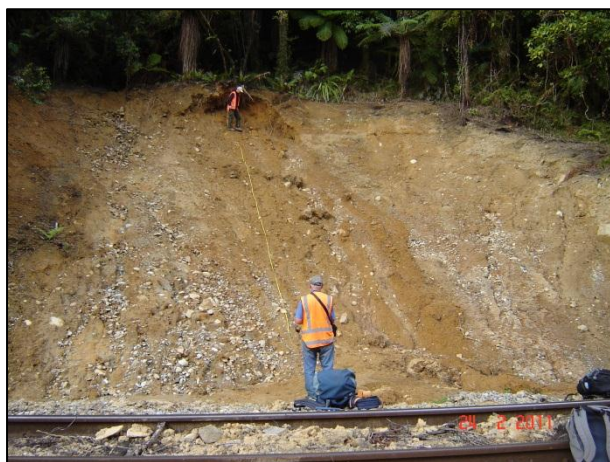
The section of rail between SNL95 and 96km is not within the project area but inclusion of landslides in this section, as observed in February 2011, is relevant to understanding the impacts from the December 2010/January 2011 rainfall events. Published geological maps for this section of the rail (Chapter 4, Section 4.4) indicate Greenland Group bedrock with overlying Kaiata Formation in close proximity to the north. Intermittent exposures of rock in the steep ( $\geq 25^\circ$ ) slopes adjacent to the rail confirmed the presence of Greenland Group bedrock from around SNL96km and towards the east. Limestone bedrock is present in the area closer to SNL97km.

Four landslides were observed between SNL95 and 97km on 24 February 2011. Three of these are shown on Figures 5.4, 5.5 and 5.6. A summary of landslide dimensions and volume estimates for each is provided in Table 5.2. The material released at SNL96.95km (Figure 5.4) comprised silty clay-bound gravels (well-graded, angular, up to 600mm maximum diameter), with some fine sand. The shallow colluvium failure did not expose bedrock at this location, but the angular blocks observed were limestone that had presumably been sourced from nearby outcrops upslope of the site.

At SNL95.60km (Figure 5.5) the colluvium failure comprised a volume of  $\sim 700\text{m}^3$  (Table 5.2) of fine silty to sandy clay with rounded alluvial gravels. Greenland Group bedrock was visible beneath the shallow (1.8m maximum depth) failure. Two failures originating on steeper (around  $50^\circ$ ) slopes occurred within very shallow ( $<1\text{m}$  depth) regolith overlying Greenland Group bedrock, as seen in the example at SNL95.30km in Figure 5.6.



**Figure 5.4: Shallow colluvial failure in clay-bound angular gravel near SNL96.95km**



**Figure 5.5: Shallow colluvial failure in clay-bound alluvial gravels near SNL95.60km**



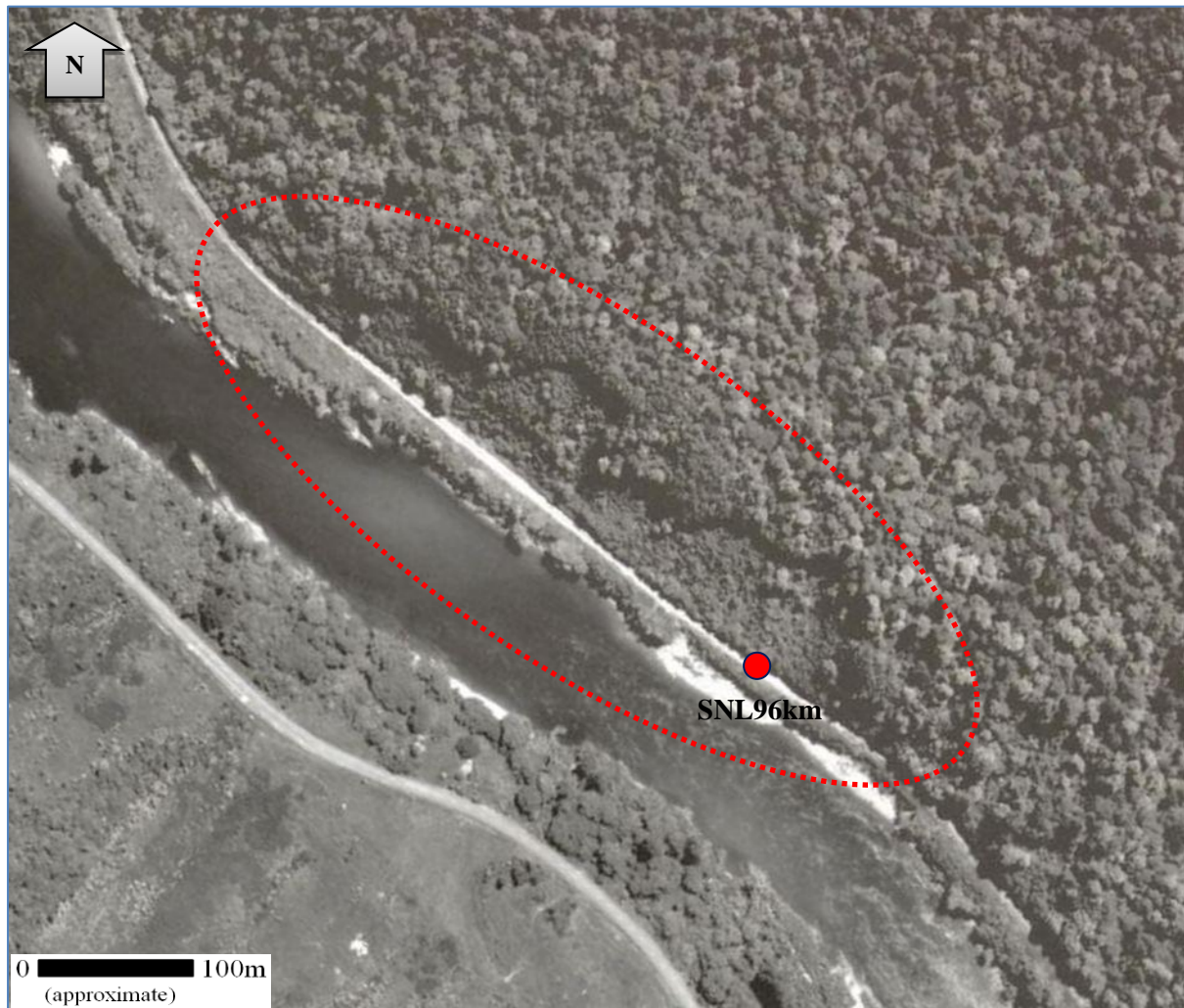
**Figure 5.6: Shallow regolith failure on Greenland Group bedrock near SNL95.30km**

**Table 5.2: Summary of dimensions and volume estimates for rainfall-induced landslides between SNL95 and 97km (H = height; W = width adjacent to base of rail alignment)**

Metrage (SNL km)	Dimensions on slope	Headscarp depth	Slope angle	Volume estimate
95.25	24m (H) x 14m (W)	$\leq 1.0\text{m}$	$50^\circ$	$\sim 300\text{m}^3$
95.30	8m (H) x 7m (W)	$\leq 1.0\text{m}$	$50^\circ$	$\sim 50\text{m}^3$
95.60	19m (H) x 21m (W)	1.8m	$27\text{-}30^\circ$	$\sim 700\text{m}^3$
96.95	22m (H) x 26m (W)	1.0m	$27\text{-}33^\circ$	$\sim 500\text{m}^3$



Previous failures in this area have been documented in Chapter 3 (Table 3.5) due to both rainfall and earthquake triggering mechanisms. The scale of previous shallow failures is visible in Figure 5.7, which also shows that natural re-vegetation is rapid ( $\leq 5$  years) along the rail corridor.

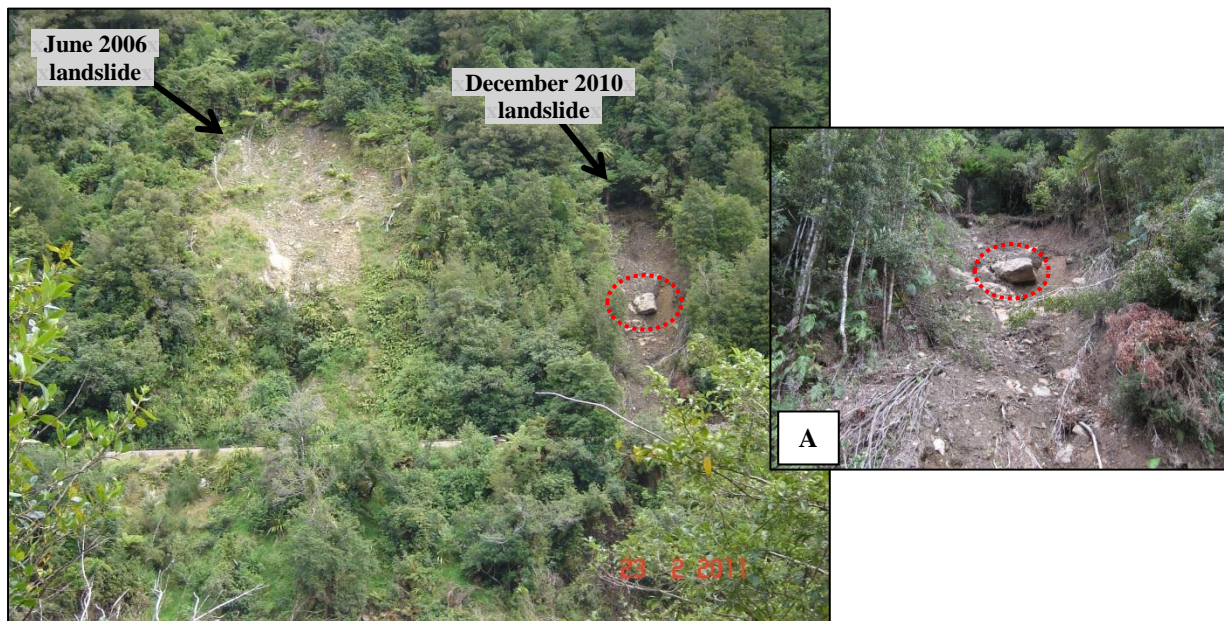


**Figure 5.7: Shallow regolith failures (circled) visible in the vicinity of SNL96km in aerial photography from 1985. Aerial photograph interpretation shows these are not related to track construction.**

### 5.5.2 SNL112.96km

Bedrock in the vicinity of SNL112.96km comprises Hawks Crag Breccia (Chapter 4, Figure 4.3). There have been many reported landslides within Hawks Crag Breccia between SNL110 and 118km (Chapter 3, Table 3.5), triggered by both rainfall and earthquakes. Previous events include large volume landslides ( $>1,000\text{m}^3$ ) that pre-date rail construction in the 1940s between SNL113 and 114km, and numerous smaller volume landslides. KiwiRail reported a train hitting a small rockfall at SNL113km, as a result of rainfall in March 2005, and a tree being struck by a train at SNL112km in August 2010.

A shallow landslide in close proximity to the feature described in Chapter 3, Section 3.4.3 (occurrence date of June 2006) was observed from SH6 on 23 February 2011 and was subsequently inspected on 25 February 2011 (Figure 5.8).



**Figure 5.8: View of partially re-vegetated June 2006 landslide at SNL112.96 (lefthand side) and recent December 2010 landslide, with a 10m<sup>3</sup> loose block of Hawks Crag Breccia (circled) near the middle of the failure. Inset A: shows a view from rail height looking up the slope.**

The dimensions of the feature on the right-hand side of Figure 5.8 from December 2010 were ~25m (height) by 5m width (adjacent to the rail). The shallow failure depth appeared variable ( $\leq 2$ m), and the slope angle averaged around 40°. The landslide is a small-scale feature with the volume of material released ~250m<sup>3</sup>. The loose block (estimated at 10m<sup>3</sup> or >25t) midway up the failure surface poses an imminent risk of dislodging and landing on, or in close proximity to, the railway.

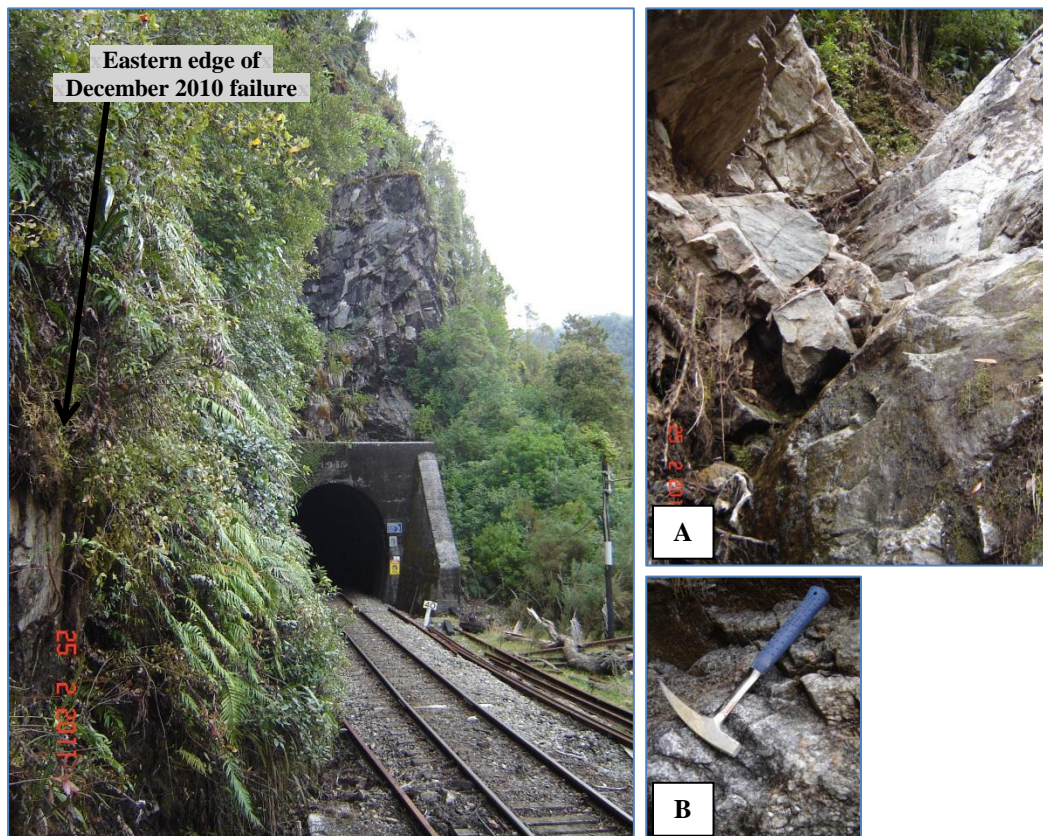
The rate of re-vegetation in the Lower Buller Gorge project area following landslide occurrences is illustrated in Figure 5.8. The 2006 failure appeared inactive, with the majority of the failure surface now covered in vegetation within the past ~5 years. Aside from the loose block visible in the 2010 landslide (Figure 5.8), the landslide is not expected to cause on-going maintenance issues, or headscarp regression, because of re-vegetation resulting in progressively increased stability over a short time period ( $\leq 5$ -years).

Hawks Crag Breccia typically has very widely spaced joints, and is a strong cemented unit as can be seen at Hawks Crag itself on SH6. The potential for blocks of several tonnes to several tens of tonnes to impact the rail corridor has to be an on-going management concern between SNL110 and 118km because of both block size and strength of the intact rock.



### 5.5.3 SNL121.35km (Cascade)

The location of a rock debris slide that occurred at SNL121.35km is shown on Figure 5.9, near the western portal of Tunnel 5. The scale of the landslide was not easy to capture from the rail alignment, and is not visible from SH6. Figures 5.9 and Insets A and B show views of the rock type (granite and gneiss), and the irregular slope failure surface on bedrock at the base of the landslide. Two distinct steps in the profile were observed for a total estimated visible slope failure distance of 50m.



**Figure 5.9:** Location of landslide at SNL121.35km near the western portal of Tunnel 5 (edge of failure surface visible on lefthand side of photograph). Insets A and B show the failure surface and the granitic/gneissic rock type.

The landslide resulted in an estimated 500m<sup>3</sup> of debris directly impacting the rail corridor. It took 2 to 3 days to clear the debris using a single excavator. An old concrete crib retaining structure was present at the base of the landslide (~4m in height from rail level), which remained partially intact. The presence of the retaining structure, and a concrete-lined drainage channel that flows into a culvert on the eastern limit of the landslide, indicates that the site has previously been problematic. These measures to retain the base of the slope and control overland water flow do not remove the risk of slope failures associated with large volumes of rock debris in storage within the catchment, and/or mobilisation under sufficiently high rainfall triggering levels.



#### 5.5.4 Discussion

Data relevant to the Lower Buller Gorge project area from WCRC (2011) included peak water levels and flow rates recorded for the Buller and Inangahua Rivers that indicated return periods of around 10 and 16 years respectively for the two catchments. The event was a medium to long duration rainfall occurrence that is not considered uncommon in the region, and it can be expected to recur at least once every 10 years with consequential impacts on rail operations.

The role of antecedent rainfall between 21 and 26 December 2010 was recognised as exacerbating the rainfall-induced landslide triggering effects on 27 and 28 December 2010. Rainfall-induced landslides triggered in the project area caused track closure and disrupted rail operations for up to one week. Roads in the area were also closed, including SH6 between Inangahua Junction and Westport.

Landslides resulting from this event in the project area involved small to medium, volumes but occurred throughout the 30km length of rail. The typical mechanisms of failure observed in February 2011, included shallow ( $\leq 2\text{m}$ ) colluvium or regolith failures, and rock debris slides comprising granitic rock sourced from natural slopes within steep catchment areas above the rail corridor.

Other landslides and geotechnical problems of typically small volume ( $<100\text{m}^3$ ) occurred throughout the project area as a result of the rainfall in December 2010 and January 2011, including subsidence of ballast and underlying foundation material at Berlins around SNL107.4km. Geoscience Consulting (NZ) Limited (2011) also noted that landsliding had occurred from upslope sources at SNL111.7, 115.9 and 118.8km.

The main management issue encountered during the response to the landslides was access to clear the debris, which is available from either end of the project area, but with SH6 closed access into the Lower Buller Gorge rail corridor was only possible from the Te Kuha end, near Westport. There is only limited vehicular access for earthmoving machinery to the Lower Buller Gorge, and it is not feasible for rail-mounted equipment to access more remote sites under such conditions.

Limited data regarding rainfall duration and intensity is available for the project area that resulted in landslide initiation, but the following examples from the inventory produced by DTec Consulting Limited (2002), as discussed in Appendix 3.1, are applicable:

- 29-31 March 1975: Westport, 80mm rainfall recorded, Buller Gorge (130mm in 24-hours). Large number of landslides.
- 2-4 July 1967: Inangahua (73mm). Few minor landslides in the Buller Gorge.
- 26-27 April 1966: Westport (63mm), Inangahua (51mm in 24-hours). Landslide closed the Buller Gorge railway near Rahui.

- 16-19 February 1955: Inangahua (90mm in <48-hours). Train struck a landslide (no mention of location) in the Buller Gorge.

Each of the above entries represents a rainfall event that resulted in numerous landslides in the Buller Gorge. Any site-specific rainfall trigger levels developed for the project area will be less than these (i.e. 50mm in 24-hours is the minimum reported rainfall intensity for slope instability), and the role of antecedent rainfall requires consideration. The absence of rainfall gauges throughout the 30km length of rail is currently the main limiting factor in establishing reliable thresholds.

## 5.6 Earthquake-generated landslides

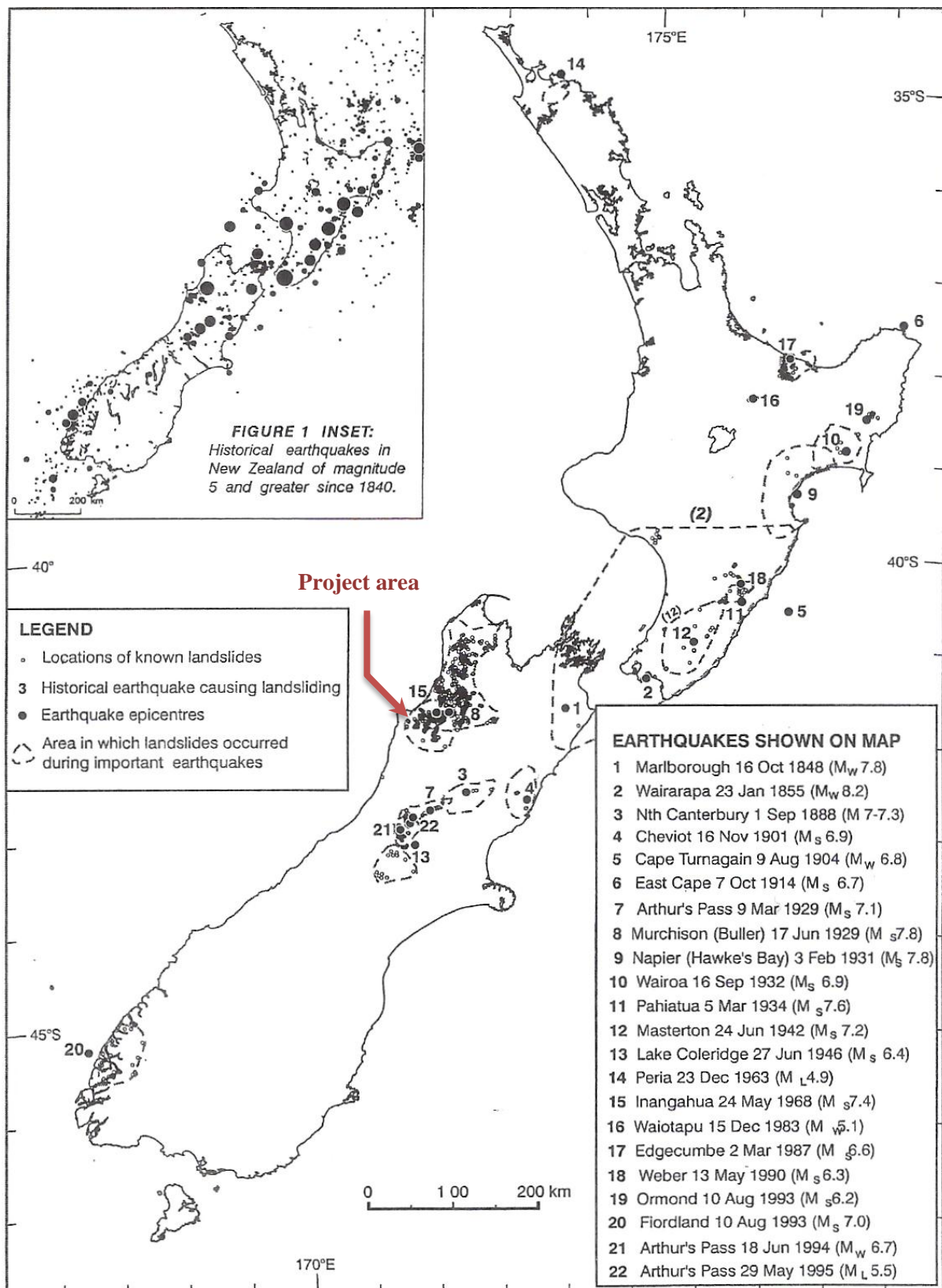
It is recognised that the rail corridor is located in a seismically active area characterised by reverse faults, and that the Lower Buller Gorge is located only ~50km from the Alpine Fault. Impacts from the two most recent large magnitude earthquakes in 1929 and 1968 are summarised in Appendix 3.3 and 3.4. This section outlines recent research regarding earthquake-generated slope movement relevant to the New Zealand setting to determine likely future frequency of earthquake-generated landslides within the Lower Buller Gorge.

### 5.6.1 Earthquake-induced landslide distribution in New Zealand

Previous studies regarding the number and distribution of landslides induced by seismic ground motion indicate that the concentration of events can be up to 50 per square kilometre (Wyllie and Mah, 2004). In New Zealand, the distribution of known landslide events triggered by seismic activity between the period 1840 and 1997 is shown on Figure 5.10 (from Hancox et al, 2002).

The distribution of landslide events in the vicinity of the project area is highlighted on Figure 5.10 and also shows the close proximity of the 1929 Murchison (Buller) and 1968 Inangahua earthquake epicentres (labelled 8 and 15 on Figure 5.10 respectively). The inset on Figure 5.10 shows the distribution of magnitude 5 and greater earthquakes in New Zealand between 1840 and 1997 and the list on the right-hand side provides basic details on the earthquake location and respective magnitudes.

Within the South Island of New Zealand the map on Figure 5.10 would look quite different based on seismic activity in the last fourteen years. Earthquake-generated slope movement occurred as a result of the magnitude 7.8 earthquake in Fiordland in 2009 and more recently in Canterbury as a result of three earthquakes measuring 7.1, 6.3 and 6.3 in magnitude on 4 September 2010, 22 February 2011 and 13 June 2011 respectively. A map showing the distribution of magnitude 6 and greater earthquakes in the South Island during the period January 1997 to August 2011, and associated summary table, is provided in Appendix 5.2.



**Figure 5.10:** Epicentres of historical earthquakes that resulted in substantial landslides within New Zealand. Dashed lines indicate the areal extent of landslide-affected regions (based on Hancox et al, 2002).

Hancox et al (2002) indicate a minimum magnitude of 5 is required for earthquake-induced rock slides or falls in New Zealand, with significant landslides expected in the event of magnitude 6 or greater earthquakes. A case study regarding the 1929 Murchison (Buller) Earthquake was presented by Hancox et al (2002), with the main summary points:

- Landslides predominantly occurred over an area of  $\sim 4,500\text{km}^2$ , extending  $\sim 90\text{km}$  north of the epicentre but only  $20\text{km}$  to the south. This delineation is shown on Figure 5.11 and does not include the project area. The total area impacted by landslides of all scales was  $\sim 7,000\text{km}^2$ .
- Five landslides are mapped adjacent to SH6 in the Lower Buller Gorge (Figure 5.11), including one that appears to correlate to Whitecliffs (south bank of the Buller River). It was noted by Hancox et al (2002) that landslides in the Lower Buller Gorge blocked the road for a few days.
- Landslides were most frequent on steep slopes (greater than  $20^\circ$ ) and the susceptible geological units included:
  - Granitic and older sedimentary rocks;
  - Tertiary mudstone, sandstone, limestone, calcareous siltstone, conglomerate; and
  - Pleistocene gravels.
- The comparatively larger and most frequent landslides originated from dip slopes in Tertiary sandstone and mudstones, and in weathered, closely-jointed granite.

The maximum areas that are likely to be affected by landslides for different earthquake magnitudes are listed by Hancox et al (2002) as:  $100\text{km}^2$  (M5);  $500\text{km}^2$  (M6);  $2000\text{--}3000\text{km}^2$  (M7);  $7000\text{km}^2$  (M8); and up to  $20,000\text{km}^2$  for M8.2. It is recognised that different magnitude scales may have been involved with different earthquakes ( $M_L$ ;  $M_S$ ;  $M_W$ ). The impacted area for the 1968 Inangahua Earthquake that resulted in landslides was stated by Hancox et al (2002) as  $3,200\text{km}^2$  ( $M_s 7.4$ ).

### 5.6.2 Slope thresholds

From the distribution of the earthquake-induced landslides recognised by Hancox et al (2002), the following two tables summarise natural slopes and rock types (Table 5.3); and typical slope thresholds for the predominant types of slope failure (Table 5.4). These are New Zealand-specific, and provide useful guidelines for future earthquake-initiated slope instability in the Lower Buller Gorge.

Table 5.3 shows that only 10% of all earthquake-initiated landslides occur on slopes less than  $25^\circ$  and that 90% can be expected to occur on slopes of  $26^\circ$  or steeper. Given that the majority (approximately 75%) of slopes within the Lower Buller Gorge are steeper than  $20^\circ$  (Chapter 4, Figure 4.2), widespread slope instability could be anticipated from an earthquake of magnitude 6.0 or greater centred within  $30\text{km}$  of the rail corridor. This is consistent with the effects of the 1968 Inangahua Earthquake, where numerous slope failures (including debris slides and rockfalls) impacted rail operations.



**Table 5.3: Natural slopes and rock types affected by earthquake-generated landslides in New Zealand (from Hancox et al, 2002)**

Slope range	Approximate % of landslides*	Typical slope and rock types
0-10°	<1%	Few failures, several low angle dip slope slides in Tertiary mudstone.
11-25°	10%	Many dip slope slides in interbedded Tertiary sandstone and mudstone, and limestone.
26-35°	30%	Dip slope failures in bedded Tertiary rocks (as above); steeper slopes in hard rocks (greywacke, schist, granite etc).
36-45°	40%	Steep cliffs, escarpments, and gorges in Tertiary limestone; scarp slopes in hard rocks (greywacke, schist, granite etc).
>45°	20%	Steep cliffs, scarps, gorges in Tertiary sediments, greywacke, schist, granites etc, especially in steep glaciated and alpine areas.
<p>Note: Comparatively smaller failures from gravel banks, terrace edges, road, railway, and other cuttings are not represented in the % values.</p> <p>* Earthquake-generated landslides</p>		

**Table 5.4: Typical slope threshold levels for main types of earthquake-generated landslides in New Zealand (from Hancox et al, 2002)**

Landslide type	Occurrence characteristics (refer table note for volumes)
Rock and debris falls	Very common, very small to large. Minimum slope angle 40°.
Rock and debris slides	Very common, very small to large. Minimum slope angle 25-35°. Less common on slopes <20.
Rock and debris avalanches	Very common, moderate to very large. Minimum slope angle 25°, more commonly 35-40° or greater. Minimum slope height 150m.
Rotational slides	Moderately common. Minimum slope angle 15°.
Rock block slides	Uncommon, large. Minimum slope angle 15°.
Mudflows (slow earthflows)	Uncommon. Minimum slope angle 10°.
Rapid soil flows	Relatively common overseas, but not in NZ. Liquefaction flows 2°.
<p>Landslide volumes referred to are: Very small (&lt;10<sup>3</sup> m<sup>3</sup>); Small (10<sup>3</sup>-10<sup>4</sup> m<sup>3</sup>); Moderate (10<sup>4</sup>-10<sup>5</sup> m<sup>3</sup>); Large (10<sup>5</sup>-10<sup>6</sup> m<sup>3</sup>); Very large (1-50 x 10<sup>6</sup> m<sup>3</sup>); Extremely large (&gt;50 x 10<sup>6</sup> m<sup>3</sup>). Note that these volume categories differ from the landslide volumes adopted for this thesis.</p>	



Table 5.4 indicates that rock and debris falls are very common on slopes steeper than  $40^\circ$ , whereas rock and debris slides typically occur on minimum slope angles of  $25\text{--}35^\circ$ . Rock and debris avalanches could occur from steeper ( $>35^\circ$ ) slopes where the height of fall exceeded 150m, but there is no evidence for these types of slope failure in previous earthquakes (including the 1968 Inangahua Earthquake). Translational debris and rock slides are considered the most likely types of future earthquake-induced landsliding in the Lower Buller Gorge rail corridor, but high vertical or horizontal accelerations ( $>1.0g$ ) could be expected to initiate falls from steeper ( $>35^\circ$ ) slopes.

It is noted that the logarithmic volume classification shown in Table 5.4 differs from that used in the present study. Hancox et al (2002) define very small earthquake-initiated landslides as being less than  $1,000\text{m}^3$  in volume, whereas this volume range includes very small, small and medium landslides as used in the present study (Chapter 4, Figure 4.2). The reason for this is that from a railway management perspective, debris even less than  $100\text{m}^3$  in volume could easily cause a derailment if the train was not able to stop in time.

### 5.6.3 Seismic sources

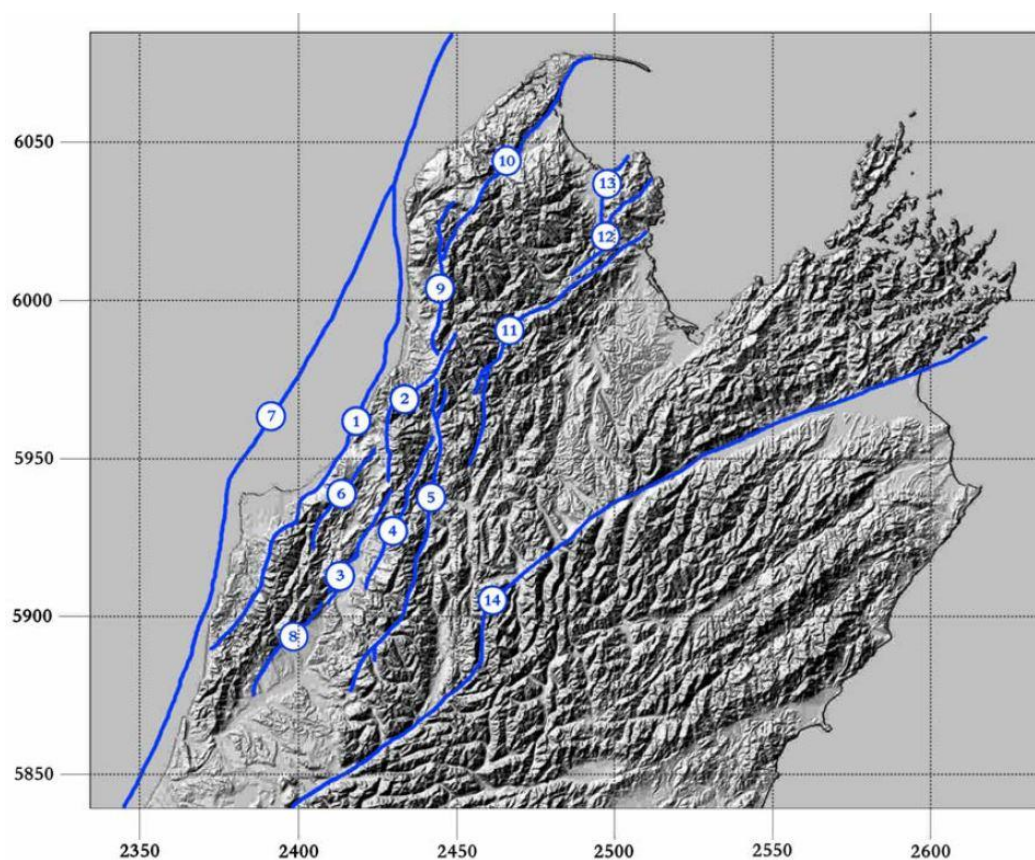
Recent studies by Stafford (2006) and Stafford et al (2008) looked specifically at the development of a seismic source model for the Buller and Northwest Nelson region to be used in probabilistic seismic hazard analyses (PSHA). One of the goals of the study was to determine whether the observed historic record of seemingly high seismic activity in the region is representative of longer term earthquake activity. The methodology used in the study included a review of previous research, which generally considered that the observed seismic rates are anomalously high for the region, coupled with consideration of all available information sources that could provide bounding criteria for long-term seismic rates (Stafford et al, 2008).

Information sources for the study included geodetic analyses, plate-motion modelling, structural geology, paleoseismic data, tree-ring analyses, precarious rock information, observed seismicity and fundamental mechanics. Fourteen fault sources were incorporated into the Buller and Northwest Nelson seismic source model by Stafford et al (2008), as shown in Table 5.5 and Figure 5.12. The study concluded that the historic record of observed seismicity in the Buller and Northwest Nelson region can be used for modelling future earthquake occurrence, and that previous research in this area has overestimated the degree of uncharacteristically high seismic rates.

The historic record shows three principal earthquakes since 1929 affecting the Lower Buller Gorge, these being Murchison (Buller) in 1929, Inangahua in 1968, and Westport in 1991. All produced magnitudes in excess of 6.0, and resulted in damage to road and/or rail facilities, allowing for the fact that the railway was not completed between Inangahua Junction and Westport in 1929.

**Table 5.5: Fault sources and geometrical parameters (based on Stafford, 2006)**

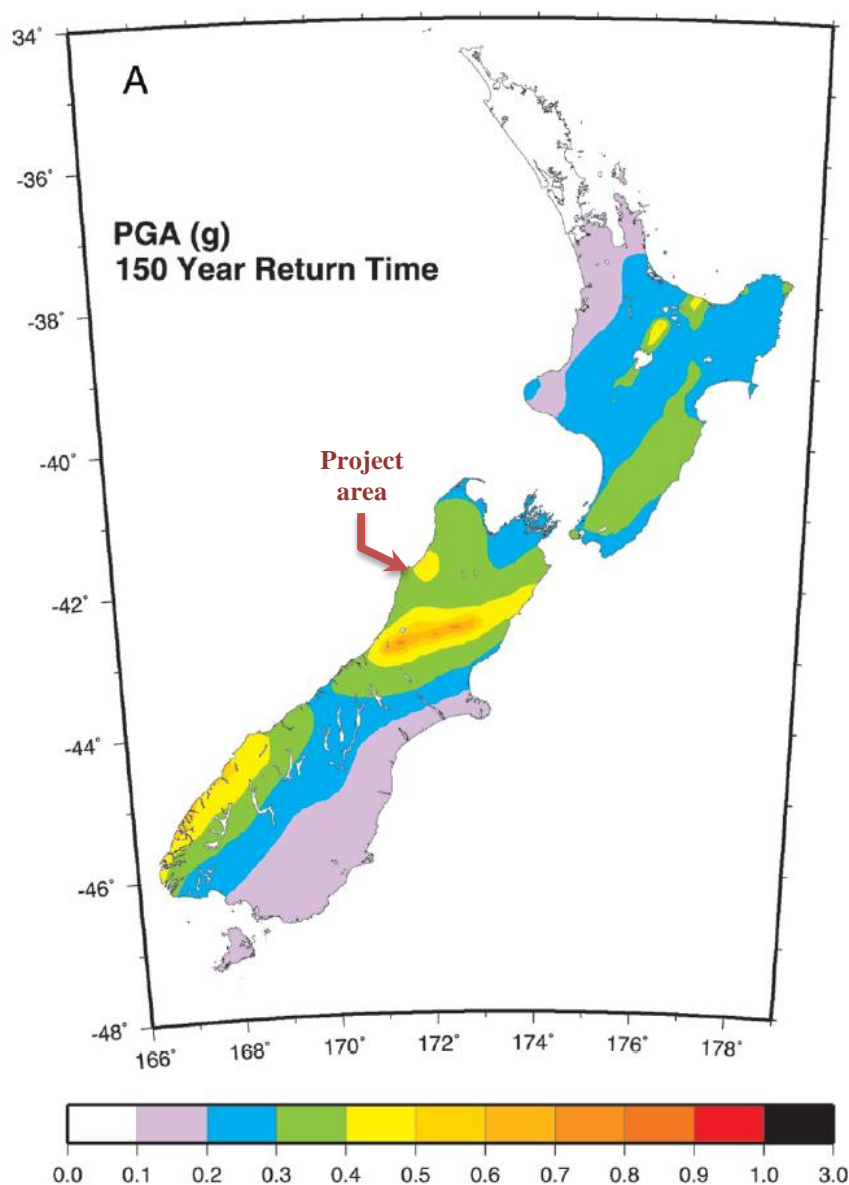
Fault Number and name (refer Figure 5.12)	Number of segments and total segment length (km)	Fault dip (°)	Fault area (km <sup>2</sup> )
1. Kongahu	5 segments, 156.5km	60	3390.0
2. Glasgow	4 segments, 47.7km	60	1005.2
3. Inangahua	2 segments, 44.7km	125	974.9
4. Lyell	1 segment, 51.9km	75	967.1
5. White Creek	3 segments, 89.5km	75	1756.3
6. Mt William	1 segment, 35.1km	73	367.0
7. Cape Foulwind	4 segments, 189.0km	65	3801.0
8. Maimai	1 segment, 25.2km	125	553.5
9. Kohaihai	2 segments, 47.5km	60	995.1
10. Wakamarama	2 segments, 72.3km	75 and 120	1506.3
11. Karamea	3 segments, 85.6km	60	1785.1
12. Pikikiruna	3 segments, 63.9km	75	1256.1
13. Pisagh	2 segments, 16.5km	75	296.4
14. Alpine	7 segments, 197.9km	45	5172.4



**Figure 5.12: Fault sources mapped in the Buller region (Stafford, 2006) used for developing a seismic source model. Numbered faults are detailed in Table 5.5. New Zealand Map Grid (NZMG) coordinates are shown with units of kilometres.**

An updated probabilistic seismic hazard analysis (PSHA) for New Zealand was presented by Stirling et al (2002). New methods were applied by Stirling et al (2002) for the treatment of historical seismic data that were combined with geologic data for 305 major active earthquake faults. Maps produced included peak ground accelerations (PGA) and 5% damped response spectral accelerations that were expected for return periods of 150, 475 and 1,000 years at average soil sites (defined as Class B site conditions in Standards New Zealand, 1992).

The 150-year return period probabilistic seismic hazard map is shown in Figure 5.13. The project area is indicated in Figure 5.13, which shows PGA values of  $\geq 0.3$  to  $\leq 0.5g$ , which are considered realistic for regolith-type failures.



**Figure 5.13: Probabilistic seismic hazard map for New Zealand from Stirling et al (2002) for site class B (intermediate soil), showing expected peak ground accelerations with a return period of 150 years (10% probability in about 10 years).**

## 5.7 Summary

Chapter 5 has focused on the ‘hazard’ component of the Landslide Risk Management framework, particularly with the nature of the upslope-sourced landslide hazards affecting rail operations along the 30km corridor through the Lower Buller Gorge. Examples were provided of the geological controls and spatial distribution of slope instability, and of the temporal effects related to specific storms including the importance of antecedent moisture conditions. The principal types of landslide hazard recognised in the Lower Buller Gorge are:

1. Rock and debris falls from very steep ( $\geq 35^\circ$ ) bedrock faces triggered by either earthquake or extreme rainfall;
2. Debris slides associated with regolith or colluvial failures from moderately steep to steep ( $> 20^\circ$ ) slopes along most of the rail corridor; and
3. Channelised debris flows in steep catchments due to bank erosion or landsliding into the watercourse.

In addition to the topographic control of instability in the steep-sided Lower Buller Gorge for the majority of the total length (30km), the main basement units (Tuhua granites; Hawks Crag Breccia; Greenland Group) all source debris slides and flows along much of their outcrop length with few specific areas identified of preferential slope failure. Tertiary bedrock units (Kaiata Formation; Whitecliffs Limestone) show localised block failures on bedding where these rock mass defects daylight or dip out of the face towards the rail. Between SNL98km and 104km Quaternary alluvium is present on the floodplains of the Mackley River and Muddy Creek, with no associated slope instability issues.

The temporal case study has represented a 1 in 10-year recurrence interval storm event that caused landslide-induced damage and track closure. This event is regarded as typical of the intermediate return period storm event that can be anticipated in a deeply incised gorge with steep sides and an annual rainfall generally exceeding 2,000mm.

Landslide damage and track closure is expected from earthquakes of magnitude 6.0 or greater centred within 30km of the rail corridor. It is noted that limited landsliding can occur in the event of a magnitude 5 earthquake (Hancox et al, 2002). Storm events producing rainfalls of 20mm or more in a 24-hour period can be considered to be landslide-triggering (Glade et al, 1998), but specific study of rainfall triggering levels is required in the Lower Buller Gorge. Particular attention needs to be focussed on antecedent moisture conditions in determining a rainfall trigger level. The absence of any rain gauges within the Lower Buller Gorge or catchments is identified as a future rail management issue to be addressed.

## CHAPTER 6: WHITECLIFFS

### 6.1 Introduction

Whitecliffs is a prominent feature of the landscape in the Lower Buller Gorge with massive limestone outcrops on both sides of the Buller River. A lot of attention has been given to the outcrop adjacent to SH6 after the fatality that occurred as a result of the 1968 Inangahua earthquake-generated rockfalls and slides. In comparison, literature reviews and discussions with rail personnel have provided very limited information regarding the limestone outcrop adjacent to the rail alignment, including any rationale, timing or justification for the 25km/hour speed restriction between SNL97.51 and 97.53km.

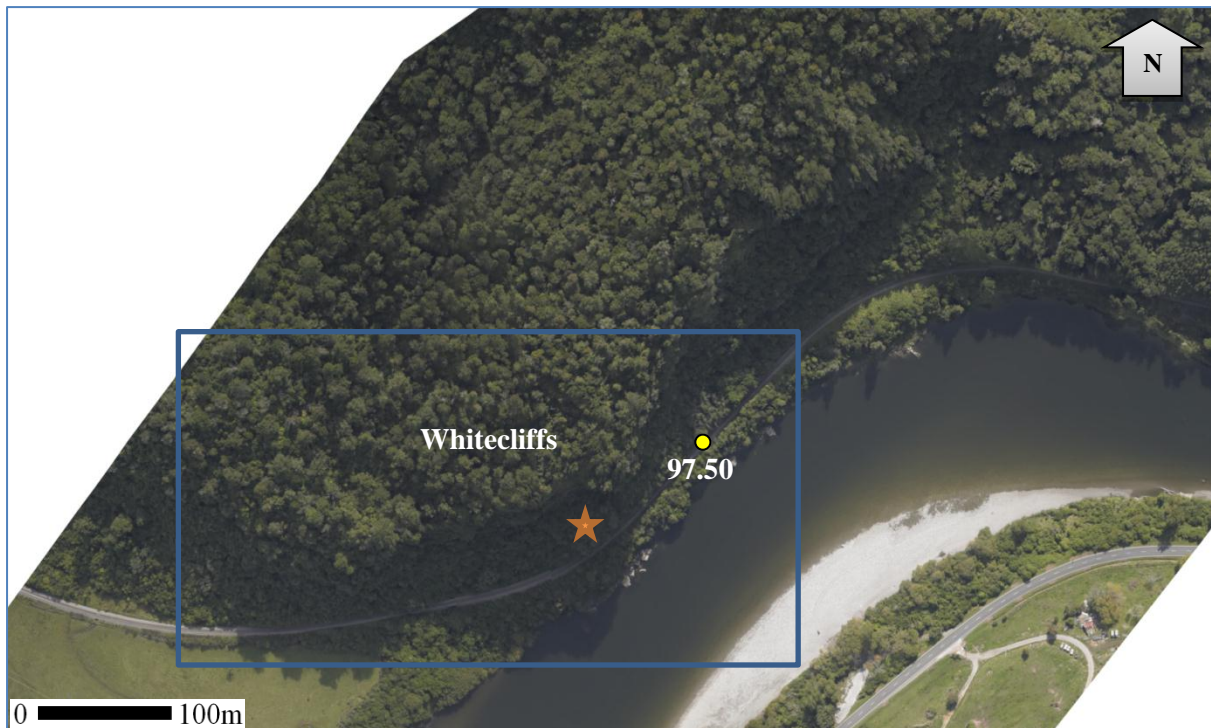
The objective of Chapter 6 is to describe the physical characteristics of the limestone cliffs adjacent to the rail corridor for the purpose of assessing any geotechnical risk associated with the removal of the 25km/hour speed restriction. The extremely short section of rail (20m) specified in the Ontrack Rail Operating Procedures, Section L6L: page 15 that has the speed restriction imposed is inconsistent with observations made on site. It is assumed for the purpose of the assessment in Chapter 6 that the area of concern is as depicted on Figure 6.1, and is approximately 500m within the boxed area, which encapsulates the rail corridor adjacent to the cliffs, and zone of limited visibility, between SNL97.40 and 97.90km.

Chapter 6 is structured into the following sections to provide a comprehensive overview of the physical, geological and mechanical rock properties at Whitecliffs, including a historical review summary of available information regarding rockfall occurrences, and triggering mechanisms:

- **Section 6.2:** Geology – based on the depositional history, structure, plan views, and cliff-face profiles.
- **Section 6.3:** Rock material characterisation – strength, elasticity, density, porosity and thin section study.
- **Section 6.4:** Slope stability assessment – qualitative evaluation from historic record, aerial photographs, and triggering mechanisms.
- **Section 6.5:** Future management considerations and speed restriction zone.

Information for this chapter was obtained from a variety of sources, including literature reviews, aerial photographs, geological and structural mapping, laboratory testing of selected samples, and preparation of thin sections. Light Detection and Ranging (LiDAR) survey data was used as the base for a number of figures in the chapter. The methodology and output of the survey is described in the following section.





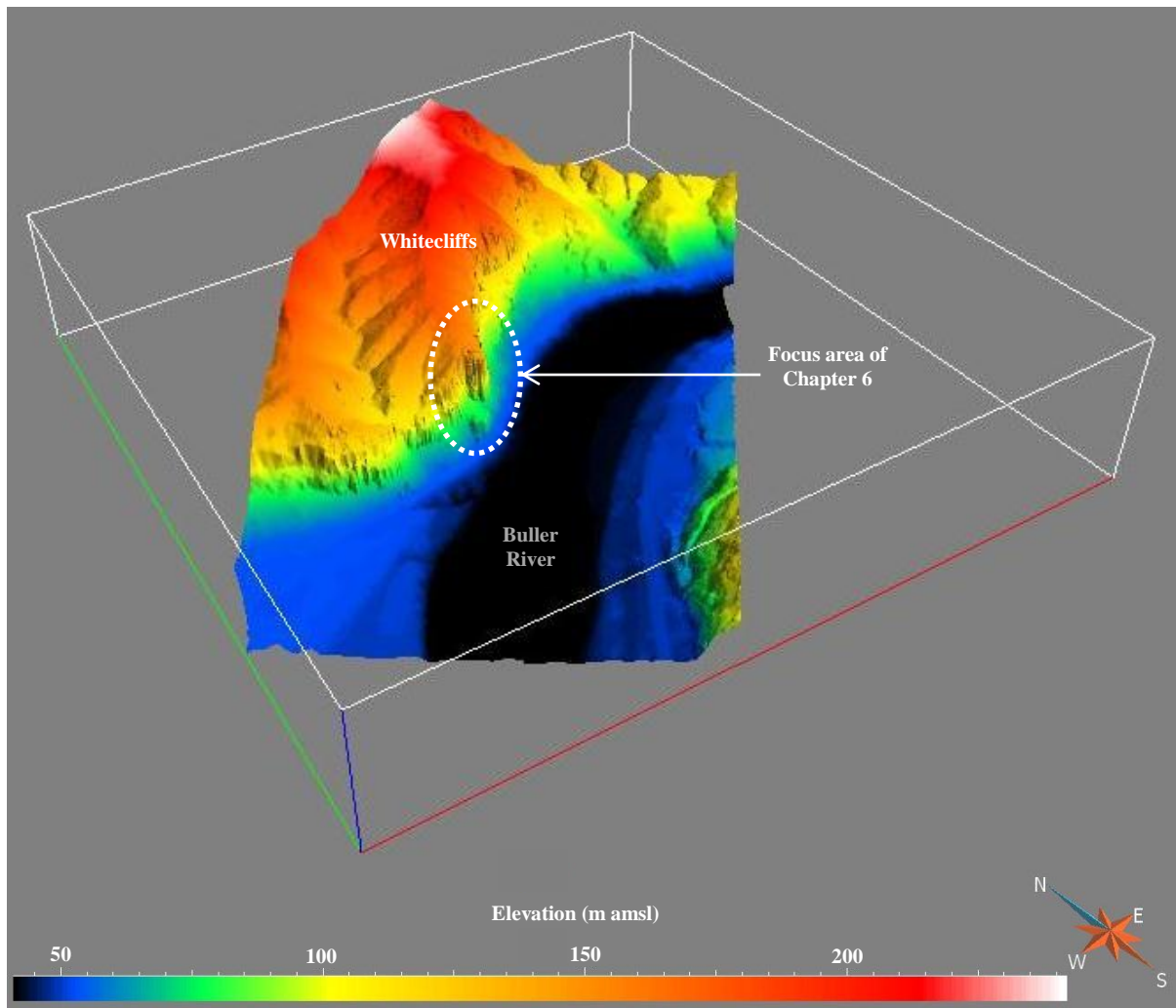
**Figure 6.1: Aerial view of Whitecliffs showing the area adopted for the risk assessment and SNL97.50km metrage location (aerial base from NZ Aerial Mapping Ltd). Star indicates the location of a very large ( $\sim 20,000\text{m}^3$ ) displaced limestone block immediately adjacent to the rail corridor around SNL97.60km.**

### 6.1.1 LiDAR survey

KiwiRail commissioned NZAM to conduct a LiDAR survey for Whitecliffs in March 2010. LiDAR is considered as a cost effective alternative to ground surveying for medium to large scale terrain modelling projects. The ground surface is characterised by collecting dense sets of elevation points by light transmission onto the target area, which then gets reflected back to the instrument being used for analysis. Additional background information regarding the LiDAR survey conducted at Whitecliffs is provided in Appendix 6.1.

LiDAR survey data has enabled visualisation of the ground surface at Whitecliffs without the interference of vegetation. A 3D-model of Whitecliffs was developed by Dr. Rouwen Lehné using the LiDAR data, and gOcad software from Paradigm, to illustrate the elevation and topographical features of the limestone outcrop (Figure 6.2). Data limitations included a number of points incorporated into the LiDAR by NZAM that represent either single rocks or vegetation features near the cliff edge. This has resulted in a pillar-type effect down the near vertical face in the 3D-model. Despite the limitations, it is clear that the prominent feature focussed on in Chapter 6 (circled on Figure 6.2) is the most probable area for failure in the event of a large magnitude earthquake, particularly when taken into consideration with geological features and rock composition discussed in Sections 6.2 and 6.3. The  $\sim 20,000\text{m}^3$  displaced block is clearly depicted in Figure 6.2 near the bottom of the outlined focus area for Chapter 6.





**Figure 6.2: Three-dimensional representation of Whitecliffs based on LiDAR survey data supplied by NZAM**

The top of the near vertical cliffs above the rail is at an elevation of 140m amsl, with the rail alignment around 55m amsl. Widely spaced ( $>10\text{m}$ ) sub-vertical fractures are visible in the cliff-face, both parallel to and normal to the face aligned roughly parallel to the rail (Chapter 3, Figures 3.1 and 3.2). Bedding planes, numerous ledges and vegetation are visible down the vertical profile (Chapter 1, Figure 1.6). A well-established talus apron is present from around 75m amsl, sloping on average at  $35^\circ$  to  $\sim 50^\circ$  down to the rail alignment. The very large displaced block (in the order of  $20,000\text{m}^3$ ) immediately adjacent to the rail at SNL97.60km provides an indication of the scale of previous failures, and its location is indicated by the star on Figure 6.1.

## 6.2 Geology

Previous studies relevant to the geological profile at Whitecliffs, and surrounding area, include Anderson et al (1994), Carter et al (1962), Lever (2001) and Lindqvist (1972). References to Whitecliffs in these papers generally consider the exposure on the southern side of the Buller River,

adjacent to SH6. Published geological maps for the area, detailed in Chapter 2 (Section 2.4) variously map the outcrop at Whitecliffs as the following stratigraphic units:

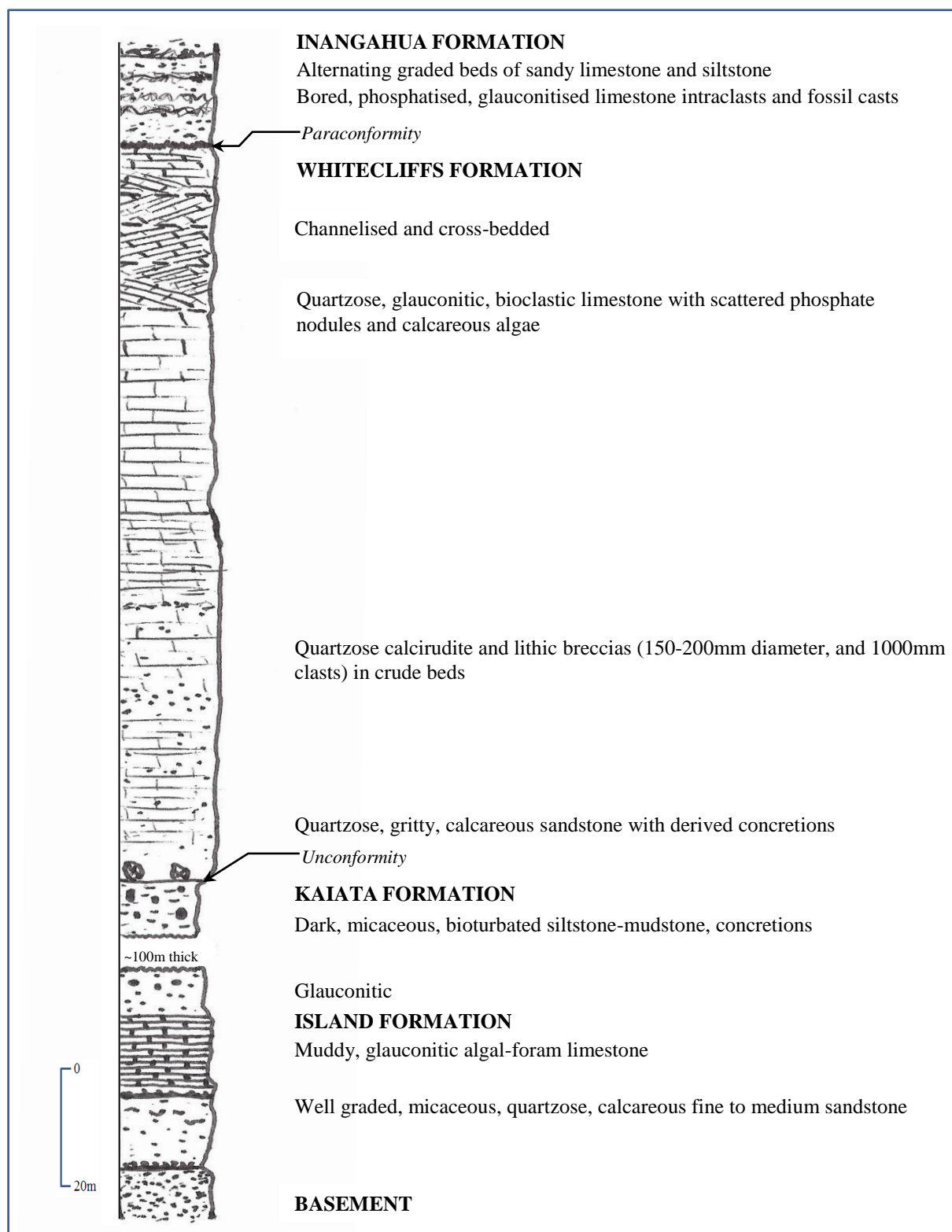
- **Nile Group:** Limestone, predominantly shallow-water bioclastic varieties (Nathan et al, 2002);
- **Cobden Limestone:** Sandy slightly glauconitic limestone with basal calcareous grit overlying hard, muddy limestone with bands of calcareous mudstone (Bowen, 1964); and
- **Whitecliffs Formation:** Sandy conglomeratic limestone, massive muddy limestone, interbedded muddy limestone and calcareous mudstone (Nathan, 1978).

To establish a more accurate geological profile for the limestone outcrop adjacent to the rail corridor, and the context for the units present, the depositional history is summarised and site specific geological observations are described in the following sections.

### 6.2.1 Depositional history

Whitecliffs is located near the western margin of the Murchison Basin (Lever, 2001). A composite stratigraphic section for the area is shown on Figure 6.3, which is based on Carter et al (1982), and the sequence (starting from the basement) is summarised below:

- Basement rocks are overlain directly by Island Formation limestone and sandstone.
- A gradational contact exists between Island Formation and the overlying Kaiata Formation (mudstone), which is the highest unit of the Maruia Group present near Whitecliffs.
- Sediment deposition at Whitecliffs is interpreted to have occurred within a basin margin location during the Maruia Group transgression and later Cainozoic phases of tectonic and sedimentary activity. The Whitecliffs Formation comprises: (1) A lower breccia lithofacies, with a matrix that is predominantly lithic (poorly sorted grains of trachyte, phyllite and greywacke), and (2) an upper limestone lithofacies (primarily a sandy, glauconitic and algal biosparite sequence).
- A variety of minerals and rock fragments from basement units make up the clastic material within the Whitecliffs Formation, including quartz and biotite.
- An unconformity is inferred between the breccia lithofacies and underlying Kaiata Formation.
- The Inangahua Formation overlies the Whitecliffs Formation. The contact between these units has been interpreted as paraconformable. The outcrop adjacent to State Highway 6 shows the contact as a distinct change in slope.
- The lower part of the Inangahua Formation comprises 100-300mm thick beds, with up to 400-1,400mm beds higher up the unit.
- A minimum thickness of 300m for the Inangahua Formation has been reported, and it is overlain by a regressive sequence, including the non-marine coal-bearing Rotokohu Formation.



**Figure 6.3: Composite stratigraphic section in the vicinity of Whitecliffs (modified from Carter et al, 1982 and based predominantly on rock exposures on the southern side of the Buller River, adjacent to SH6). Kaiata Formation is the highest unit of the Maruia Group, and is overlain by Whitecliffs Formation (Cobden Group) and Inangahua Formation (Blue Bottom Group).**

As indicated in Figure 6.3, the depositional history in the vicinity of Whitecliffs involved a shallow marine transgression that was terminated in the Oligocene by a period of faulting and basin formation (Carter et al, 1982). Tilting is also inferred to have occurred around this time, resulting in a slightly angular unconformable contact between Maruia Group transgressive sediments, overlying breccias and Cobden Group limestone (Carter et al, 1982). A hardground developed in the mid to late Oligocene at the top of the Cobden Group, along a burrowed and bored surface. A period of renewed tectonic activity is inferred later in the Oligocene, associated with the deposition of mudstone.

### 6.2.2 Whitecliffs plan view

To access the top of Whitecliffs, a basic track has been marked by KiwiRail staff to enable annual monitoring of pins located across fractured limestone with some detached blocks. As summarised in Section 1.6.5, there has been no detectable movement at the monitoring locations chosen by KiwiRail in the 18 years since the first survey round was completed in 1993 (refer Appendix 1.3 for details).

The ground surface at the top of Whitecliffs is very irregular, comprising broken and semi-detached limestone blocks of variable size. Once the highest point is reached (approximately 155m elevation, refer Figure 6.4), the slope drops down towards the southeast at 35° on average to the start of the near vertical profile. Within this area of Whitecliffs there are three open fracture sets, labelled on Figure 6.4 as Fracture 1 (northernmost), 2 and 3. The features are joint-controlled but are being described in this chapter using the generic term ‘fracture’.

The fractures trend generally in the same direction (southwest to east-northeast). GPS coordinates of the fracture locations were taken and this data added into the site model for Whitecliffs using ArcGIS software. Measurements of the open fractures are summarised in Table 6.1, and extrapolated from marked locations on Figure 6.4. A representative cross-section of the fractures and cliff-face profile is provided on Figure 6.5, and its location is shown on Figure 6.4.

**Table 6.1: Description and measurements for open fractures mapped at the top of Whitecliffs**

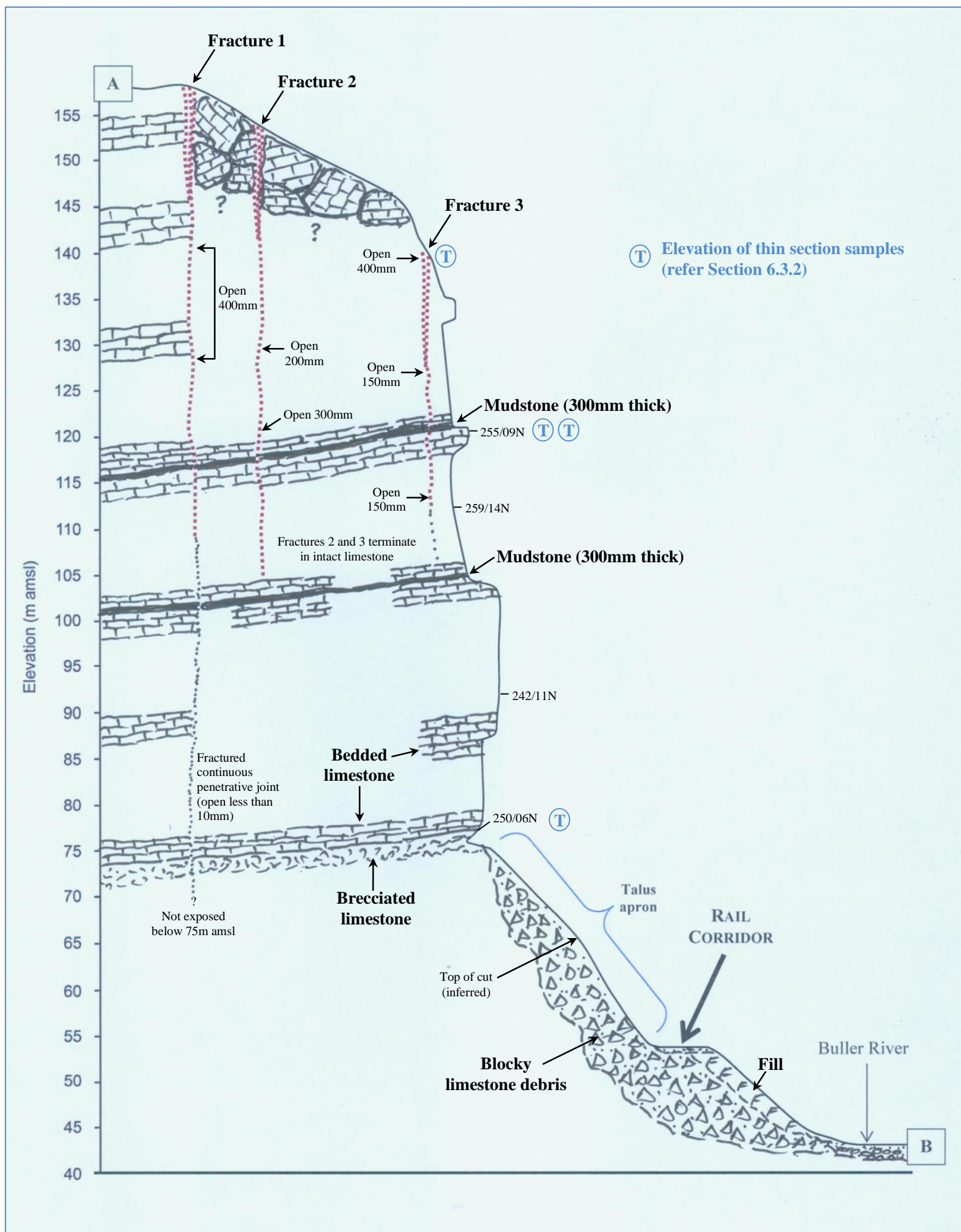
Reference*	Description and measurements
<b>Fracture 1</b>	<ul style="list-style-type: none"> <li>Western end: measured for 16m, open horizontally to a maximum of 1.4m, decreasing to between 0.3-0.6m towards the east.</li> <li>The fracture extends laterally from a true bearing of 040 (north-east), changing to a bearing of 070 (east-northeast) until definition is lost as the ground slope increases and material on the downslope side of the fracture has dropped away.</li> <li>Material has infilled the fracture but it is still open down to a maximum of 1.5m below ground level (western end).</li> <li>Vertical displacement is less than 0.2m, downthrown towards the south/southeast.</li> </ul>

Reference*	Description and measurements
<b>Fracture 2</b>	<ul style="list-style-type: none"> <li>Measured for 8m, stepping from a bearing of 060 at the western end to around 090 at the eastern end.</li> <li>Open horizontally to a maximum of 1.3m, decreasing to 0.6m towards the east.</li> <li>Maximum vertical displacement was 0.9m, downthrown towards the south. Infilling has occurred with the measured fracture depth ranging between 0.4m and 0.8m from the downthrown side of the fracture.</li> </ul>
<b>Fracture 3</b>	<ul style="list-style-type: none"> <li>Measured for 9m, orientated east to west.</li> <li>Horizontal opening decreases towards the east (from 0.9m to 0.4m).</li> <li>Maximum measured vertical displacement of 1.2m, downthrown side towards the south.</li> <li>Access to the western extent of Fracture 3 was not possible.</li> </ul>
* Refer Figure 6.4 for spatial distribution	



**Figure 6.4: Fractures mapped at the top of Whitecliffs and start of Profiles W1 and W2**





**Figure 6.5: Representative cross-section of the Whitecliffs profile (location of section A–B shown on Figure 6.4).**

There is no evidence at the top of Whitecliffs to indicate any active movement of the displaced limestone blocks and fracture sets, based on vegetation cover, the degree of infilling of the open fractures, and absence of slab failures on the sides of the open fractures. This observation is supported by measurements recorded by KiwiRail staff of metal pins that are located across the western end of Fractures 1 and 3 (Appendix 1.3). The vertical offsets across each fracture show the maximum downthrown extent increasing towards the cliff-face from 0.2m (Fracture 1) to 1.2m (Fracture 3). The tapering observed in opening widths of the fractures, to penetrative joints at greater depth below the ground surface, does also suggest past toppling movements (Figures 6.4 and 6.5).

Numerous other openings and fractures at the top of Whitecliffs were noted during the walk up to the top of the cliff. The persistence, depth and width of the fractures varied, some representing only single displaced blocks of limestone  $\leq 10\text{m}^3$ . While localised failures and instability may occur within other areas of Whitecliffs, the risk of failure, and resultant impacts to rail operations, increases with proximity of the slope to the rail alignment. For this reason, the areal extent mapped in detail for this thesis at the top of Whitecliffs is considered the highest risk area at present.

### 6.2.3 Cliff-face profiles

To confirm the stratigraphic sequence detailed by previous studies, two abseil traverses were completed down the cliff-face at Profiles W1 and W2. The start of each descent is shown on Figure 6.4. The objectives of the traverses were to determine bedding and joint orientation; identification of any bedding plane shears on which movement has occurred; dilation information on fracture sets, including any measureable offsets; and lithological variation within the limestone.

A tape measure was fixed at the start of each traverse and lowered during the descent to correlate observations with locations on the cliff-face. Measurements from the tape measure are indicative only as the cliff-face was not always vertical and allowance has been made in the following sections for slope variation based on elevation data from the LiDAR survey. All distance measurements referring to profile lengths have been converted to elevation (m amsl) in Sections 6.2.4 and 6.2.5.

Details regarding bedding attitude and thickness, ledges, and lithology have been incorporated onto Figure 6.5. A photographic record of the vertical profile was also undertaken with relevant photographs provided in the following sections. An attempt was made to abseil down approximately midway between Profiles W1 and W2, but the overhang was greater in this area, and the absence of any trees or other features to secure the ropes caused safety concerns. There was also uncertainty regarding obtaining useful data due to the large overhang.

#### 6.2.4 Profile W1

The first traverse (Profile W1) was completed by Abseil Access Limited staff only, due to health and safety constraints. The information obtained was useful in terms of observations of the sub-vertical fractures on the western side of the prominent outcrop as shown on Figures 6.2 and 6.4. Only limited lithological and structural descriptions were relayed by Abseil Access Limited during the first traverse. Profile W1 focussed on two of the main open fractures, specifically the western expression of Fractures 1 and 2. The following comments, and photographic log, are based on relayed information from Abseil Access Limited:

- Unvegetated rock was exposed from the start of the descent, near the monitoring pins (140m, Figure 6.6) to ~98m.
- Loose (detached) limestone blocks with volumes exceeding 10m<sup>3</sup> were predominant between 140m and 128m. The western extent of the detached blocks was bounded by Fracture 1 (Figure 6.7).
- Fracture 1 between 140m and 128m was described by Abseil Access Limited as near vertical, open on average to 400mm, and was estimated to persist into the rock face for at least 10m into darkness. Surface mapping in Figure 6.4 indicates continuity for at least 35m.
- The first ledge was encountered at 128m, and the limestone down to this level was very well indurated and massive. The ledge itself was estimated at 10m (lateral extent). The near vertical open Fracture 2 at this level (130m amsl) had a width of 200mm, orientated east-northeast (080) into the cliff-face (Figure 6.8).
- The first obvious lithological change down the main cliff-face below the detached blocks was reported at 125m (based on the presence of bedding planes, spaced at ~1m). A bedding strike and dip measurement was not possible at this location but the strata was noted as dipping northwards, back into the cliff-face at a shallow angle, as evident in Figure 6.9. The limestone was still very well indurated.
- A ~4m overhang was encountered at 120m, which persists around the cliff-face. This level marks the base of a detached block displaced horizontally by ~0.5m. Fracture 2 at this level was open 300mm.
- A change in lithology was noted at 116m, with a 300mm thick grey mudstone unit.
- Fracture 2 at 108m was recorded as closed.
- Thick vegetation was present from 98m onwards to the base of the descent for Profile W1, and no additional data from the cliff-face was possible below this elevation.

Limited information regarding bedding planes was obtained during the Profile W1 descent. Abseil Access staff who completed the descent commented on how structural features in the limestone below the level of the detached blocks (from 140 to 128m elevation) were consistent laterally around the face.





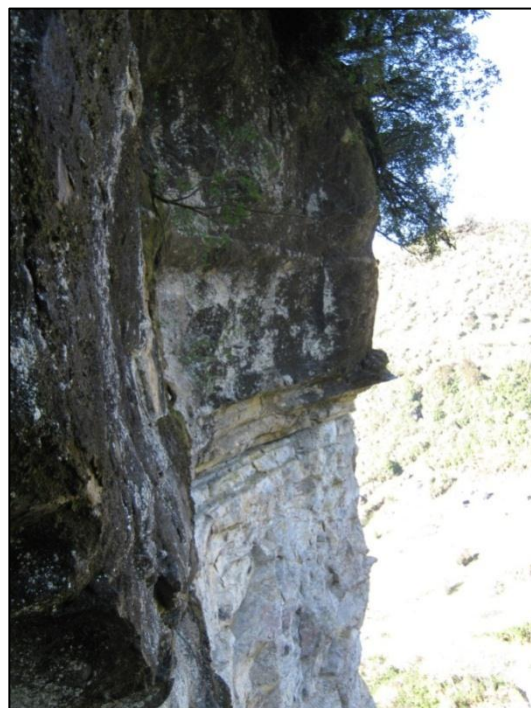
**Figure 6.6:** Start of Profile W1 descent at 140m elevation, near monitoring pin location. Limestone block on the lefthand side of the photograph is completely detached (volume of  $\sim 10\text{m}^3$ ).



**Figure 6.7:** View looking up from  $\sim 135\text{m}$  elevation during Profile W1 descent showing detached block on right-hand side of photograph and massive, highly indurated limestone.



**Figure 6.8:** View of sub-vertical Fracture 2. Open around 200mm in this example from  $\sim 128\text{m}$  elevation.



**Figure 6.9:** View looking east from  $\sim 125\text{m}$  elevation during Profile W1 descent. Bedding dips back into the face (north) at a shallow angle ( $<15^\circ$ ).

### 6.2.5 Profile W2

The second traverse (Profile W2, Figure 6.3) was undertaken by abseiling staff, Kristel Franklin and thesis supervisor Dr. Rouwen Lehné to collect more detailed geological and structural descriptions. Profile W2 focussed on Fracture 3 and results are presented in Table 6.2. The main observations made during the logging of Profile W2 were as follows:

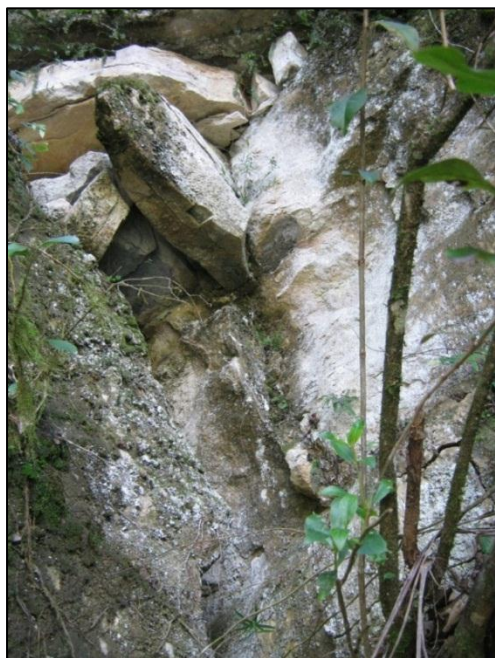
- From the start of the traverse at 140m amsl to 118m amsl the limestone is massive and strong, with layering at about 500mm spacing but with no obvious bedding planes or lithology changes.
- Over this 22m vertical distance Fracture 3 tapers from a maximum opening width of 400mm to about 150mm, and a 300mm thick mudstone unit at ~118m is the only lithology change noted.
- The massive and strong limestone above 118m is shown in Figures 6.10 to 6.12, and the partially slaked and well-bedded 300mm thick mudstone exposure in Figures 6.13 and 6.14.
- Minor slabbing releasing blocks of a few cubic metres in volume on an irregular sub-vertical and very rough fracture surface at ~119m is interpreted as a stress-release feature (Figure 6.15).
- From 118m to the top of the talus apron at 75m amsl the limestone is strong and well indurated, but bedding becomes better developed and spacing is 40-400mm around 90m amsl.
- Fracture 3 appears to terminate in intact rock at about 110m amsl (Figure 6.5), suggesting that the relatively rough fracture has formed by a toppling mechanism on a laterally continuous joint.
- A second 300mm thick laterally continuous grey mudstone is present at 105m, and some bedding release of limestone blocks about 1m<sup>3</sup> in volume becomes evident at about 85m (Figure 6.16).
- Close examination of the limestone exposed at the top of the talus apron (about 75m amsl) shows it to be brecciated, and to contain some visible glauconite as well as evidence of cross-beds.

**Table 6.2: Summary of Profile W2**

Elevation (m amsl)*	Description	Measurements (strike/dip) <sup>#</sup>
140	Start of descent, massive, very well indurated limestone close to open fracture (Fracture 3). Loose blocks overhanging (Figure 6.10). Black lichen covered the limestone from 140 to 134m (Figure 6.11).	-
139	Vertical fracture open 350mm, extends northwest into cliff-face for at least 5m, then darkness.	-
134	First ledge encountered. Layering at 500mm intervals but no obvious bedding planes. Fracture 3 open between 250mm and 400mm, loose blocks visible with the fracture, which is sub-vertical, orientated north into the cliff-face.	-
132	Descent continues over ledge, through vegetation. Overhang (~2m), no physical contact possible with cliff-face. Bedding planes observed dipping at a shallow angle back into the cliff (north), bed thickness ranged between 300mm and 800mm (Figure 6.12).	-



Elevation (m amsl)*	Description	Measurements (strike/dip) <sup>#</sup>
119	Next ledge. 300mm thick mudstone unit noted around ~118m (top of Figure 6.13 and 6.14). Localised slab failures evident (Figure 6.15). Bed thickness continues to range between 300mm and 800mm. Fracture 3 is open 150mm, vertical to sub-vertical, and persists northwards into the cliff-face. Four limestone samples collected.	250/11°N 255/09°N
112	Descent continues through vegetation from 119 to 112m amsl. Fracture 3 is near vertical, 150mm wide within closely bedded limestone.	259/14°N
105	Loose limestone blocks on ledge of variable sizes (<1m <sup>3</sup> ). Grey mudstone unit extends laterally around the cliff-face (300mm thick). Ledge encountered at 100m amsl.	-
90	Strike and dip measurements taken at 90m amsl. No contact possible with rock from 85m elevation until base of descent (at the top of the talus apron at 75m amsl). Horizontal bedding with thickness ranging between 40mm and 400mm. Figure 6.16 taken from an elevation of ~85m, showing a bedding-controlled failure surface.	242/11°N 284/05°N
75	Top of rock talus apron. Cliff-face is accessible. Limestone is very well indurated and contains numerous clasts at the base of the cliff.	250/06°N
* based on LiDAR survey data # all measurements from bedding planes		



**Figure 6.10:** Start of Profile W2, looking northwards and back up the cliff at loose overhanging rocks.



**Figure 6.11:** Massive, very well indurated limestone, looking back up at the first 6m of the descent.





**Figure 6.12: Fracture 3 (arrow) around ~128m elevation. View east.**



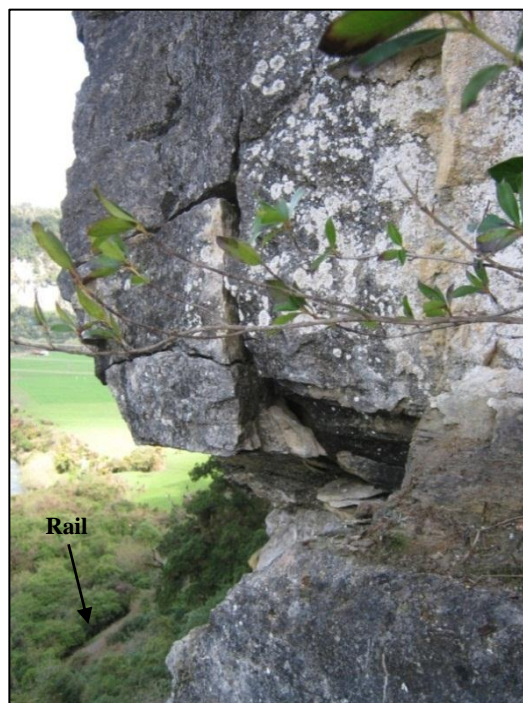
**Figure 6.13: Mudstone (300mm thick).**



**Figure 6.15: Slab failure development in weathered surface of exposed cliff-face around ~119m elevation.**



**Figure 6.14: Localised fracture observed at an elevation of ~118m. Note the finer grained mudstone near top of photograph (also shown in Figure 6.13).**



**Figure 6.16: Cliff-face profile from ~85m elevation, view west across the rail corridor (indicated by arrow).**

Bedding attitudes were measured between 119m and 75m amsl showed dips consistently into the face (to the north) at angles between 5 and 14°. Fractures 1, 2 and 3 mapped at the top of Whitecliffs represent widely-spaced persistent joint sets (~10m spacing), but these are irregular, rough and only broadly planar.

There were no bedding plane shear zones observed during the descents of Profiles W1 and W2, and only two 300mm thick mudstone units were recognised. While these will over time display preferential slaking, there was no evidence of significant undercutting initiating large-scale (10m<sup>3</sup> or greater) block release in the overlying limestone. The existing rock talus apron and the vegetation at the base of the limestone bluffs at Whitecliffs, reduce the potential for rockfalls released from the cliff-face to reach the rail corridor down the very rough blocky surface.

### 6.3 Rock material characterisation

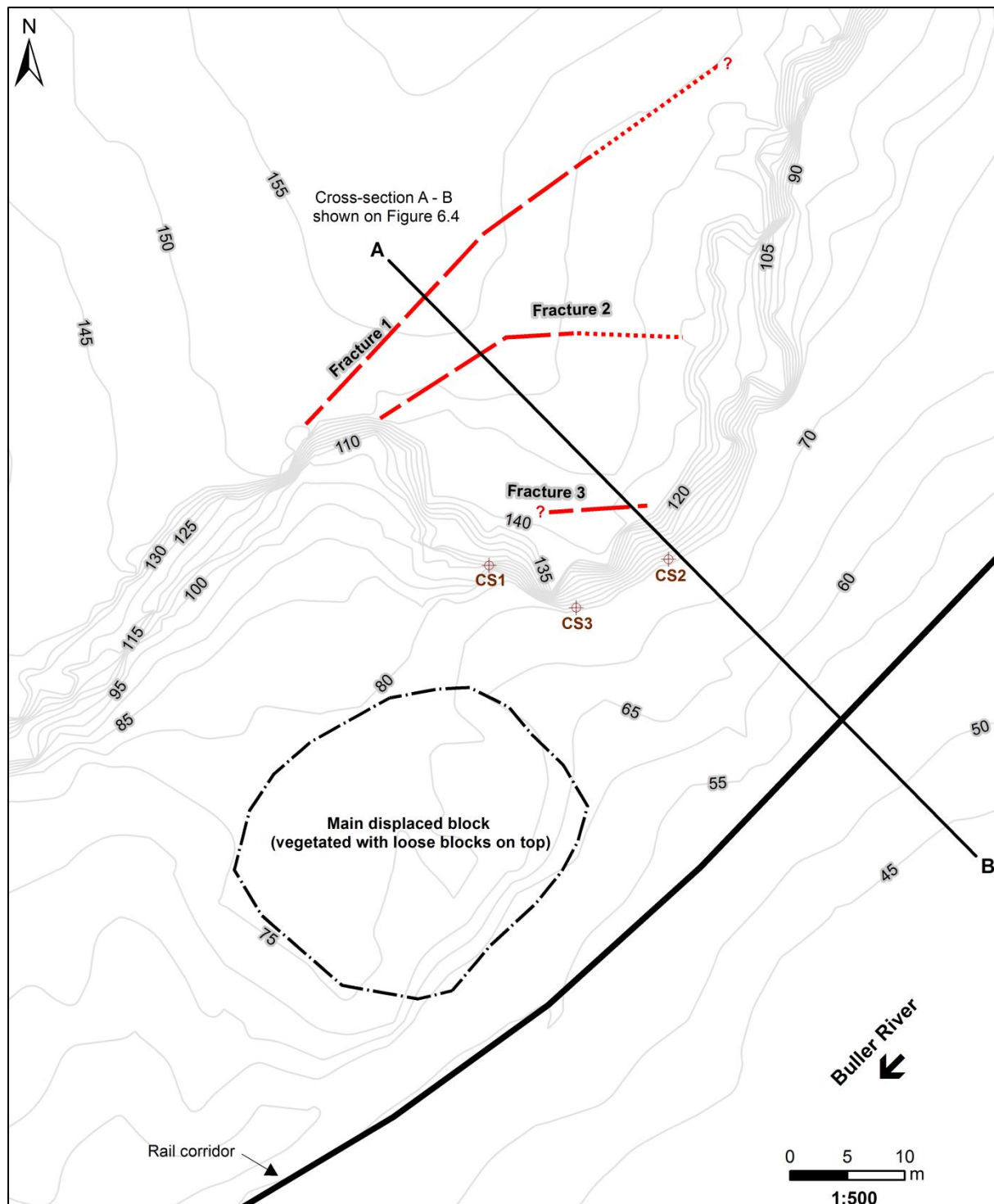
A limited laboratory testing programme was undertaken to supplement the field mapping. Samples were collected where feasible during Profile W1 and W2 descents. Due to the hard nature of the rock outcrops and access limitations (including overhanging) during abseiling only limited chip samples could be recovered. Sample references detailed below refer to the approximate elevation (m amsl) they were collected from *in situ* using a geological rock hammer:

- **Sample A.** 140m elevation (top of Whitecliffs close to the monitoring pins)
- **Sample B.** 119m elevation (Profile W2)
- **Sample C.** 75m elevation (top of talus apron)
- **Sample D.** 55m elevation. Large displaced limestone block immediately adjacent to the railway

In addition, three core samples (CS1, CS2 and CS3), with dimensions of 300mm (length) and 43.5mm (diameter) were recovered by drilling horizontally into the base of the cliffs close to where open sub-vertical fractures were identified. Core sample locations are shown on Figure 6.17. Cores were recovered by Abseil Access Limited using a portable electric diamond drill powered by a generator. Point-load strength testing was carried out on Samples A to D. Testing procedures were in accordance with ISRM Standard Methods (Ulusay and Hudson, editors, 2007). Analytical testing for the three core samples is outlined below:

- **Core sample CS1:** recovered from the sedimentary breccia at the base of Profile W1 (Figures 6.4 and 6.17). Core recovered was not intact and not scheduled for testing.
- **Core sample CS2:** recovered from a similar stratigraphic position on Profile W2. Samples were prepared for testing, including porosity, density, seismic velocity and UCS determination without strain measurement.

- **Core sample CS3:** recovered from the same stratigraphic position (close to the sedimentary breccia contact with the overlying limestone), approximately equidistant between CS1 and CS2 (Figure 6.17). The same analytical testing was conducted as for CS2.



**Figure 6.17: Location of core samples CS1 to CS3**

Test results are summarised in Tables 6.3 and 6.4. Thin sections prepared from Samples A to D were also examined, as outlined in Section 6.3.2. All testing for this thesis was carried out at the University of Canterbury with assistance of the Geological Science Department technicians to ensure best practice procedures were adhered to. The seismic analyser used is shown in Figure 6.18 (GCTS Ultrasonic Velocity Test System, ULT-100 CATs). An example of the UCS test is shown in Figure 6.19, including the resulting failure for CS3 (Figure 6.20) and CS2 (Figure 6.21).

**Table 6.3: Materials testing results from Whitecliffs**

Sample (elevation)	$I_{S(50)}$ MPa	Equivalent UCS (MPa) *	Measured UCS (MPa)
A. 140m	1.72 (8)	41	-
B. 119m	3.08 (7)	74	-
C. 75m	3.66 (6)	88	52.3 (CS2) 42.4 (CS3)
D. 55m	2.03 (5)	49	-
<p>Notes: Outliers removed for Sample A and B. Averages presented for Samples C and D (limited samples).</p> <ul style="list-style-type: none"> <li>- Core samples obtained by drilling into the outcrop at the top of the talus apron; two cores obtained.</li> <li>- Numbers in brackets represent number of valid results used (point-load tests).</li> <li>- * Conversion based on equivalent UCS = <math>24 \times I_{S(50)}</math></li> </ul>			

**Table 6.4: Materials testing results from core samples CS2 and CS3**

Parameter		Units	Core Sample CS2	Core Sample CS3
Porosity-Density	Diameter	mm	43.5	43.3
	Length	mm	41.5	38.2
	Dry mass	g	149.7	142.2
	Saturated mass	g	154.0	144.6
	Dry Mass Density	kg/m <sup>3</sup>	2429	2522
	Porosity	%	7.0	4.3



Parameter		Units	Core Sample CS2	Core Sample CS3
Elastic Parameter	Diameter	mm	43.5	43.3
	Length	mm	41.5	38.2
	P-wave velocity	m/s	4785	4364
	S-wave velocity	m/s	2511	2343
	Poisson's ratio	-	0.31	0.30
	Modulus of elasticity	GPa	40.1	35.5
Unconfined Compressive Strength	Diameter	mm	43.5	43.3
	Length	mm	110.4	118.2
	Failure load	kN	77.7	63.0
	UCS value	MPa	52.3	42.4



Figure 6.18: Seismic analyser



Figure 6.19: UCS testing



Figure 6.20: Core sample CS3 failure



Figure 6.21: Core sample CS2 failure

### 6.3.1 Interpretation of results

The mean point-load strength tests (Table 6.3) show a range in  $I_{S(50)}$  MPa values from 1.72 in the finer grained micritic limestone near the top of Whitecliffs (140m elevation) to 3.66 in the top of the underlying sedimentary breccia (75m elevation). The equivalent UCS values, using a conversion of  $24 \times I_{S(50)}$ , suggest a range from 41 to 88MPa (Table 6.3). The two core samples tested (CS2 and CS3) fall within this range at 42 and 52MPa respectively, and minimum limestone strength of 40MPa can reasonably be adopted. It is recognised that only having two strength results from the core samples is not a reliable data-set, but logistical and financial constraints did not allow for additional coring at Whitecliffs.

Porosity-density and elasticity data are reported in Table 6.4 for CS2 and CS3. These indicate a rock material density of around 2500kg/m<sup>3</sup>, and a porosity range of 4 to 7%. Seismic analyser testing gave modulus of elasticity (E) values of 35 and 40GPa, and Poisson's Ratios of 0.30 and 0.31. Both results indicate a moderately strong and elastic material. Textural data presented in Section 6.3.2 confirms that the limestone has a low porosity (~5%).

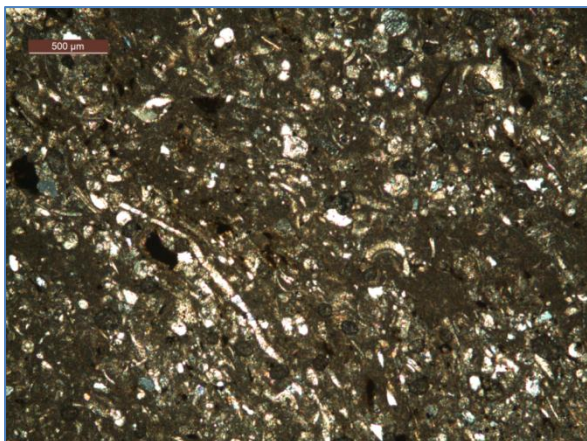
The limited testing has demonstrated that the limestone at Whitecliffs has sufficient intact rock strength for long-term stability, and there is no evidence for loss of strength by weathering or solutioning. Given the very wide joint spacing, flat-lying bedding attitude, absence of weathering, and relatively high material strength, it can be concluded that there is no likelihood of intact rock failure in the bluffs above SNL97.5km under normal gravity loading.

### 6.3.2 Thin sections and rock textures

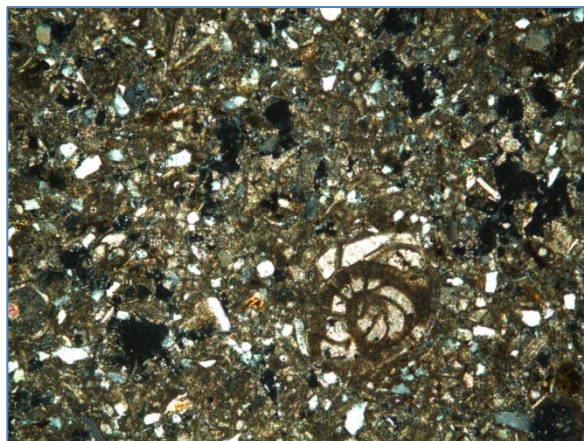
The very hard nature of the limestone rock encountered during abseiling, and restricted sampling techniques available, resulted in limited sample collection during the two descents (six samples in total). Representative rock samples were selected for thin section preparation to confirm the rock observed during the two descents; top of Whitecliffs; and *in situ* material at the top of the talus apron. A sample was also included from the large displaced limestone block adjacent to SNL97.60km. Photomicrographs and descriptions of the thin sections viewed are provided in Figures 6.22 to 6.27.

Interpretation of the depositional history at Whitecliffs, as deduced from a limited number of samples and thin sections from Profile W2, has been developed with assistance from Dr Catherine Reid, Geological Science Department, University of Canterbury.

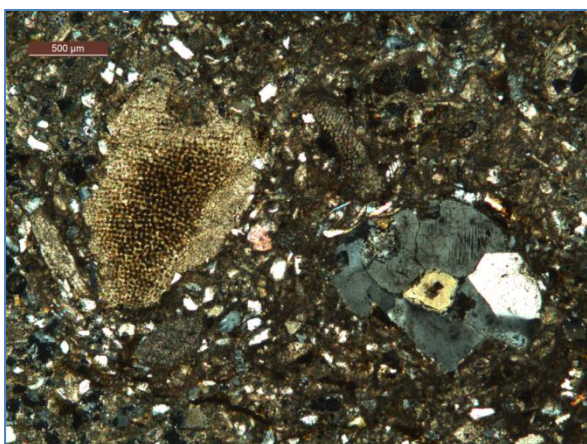




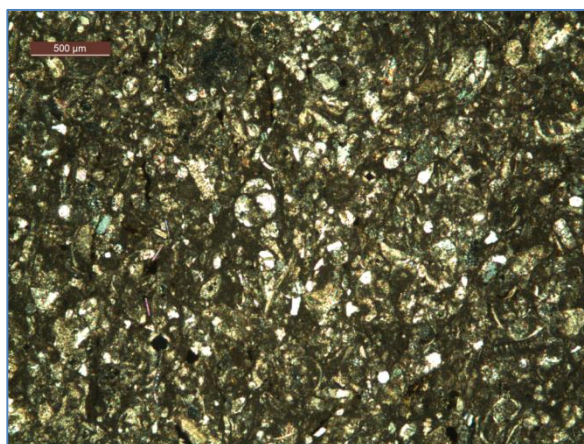
**Figure 6.22:** Thin section of fine-grained deep-water limestone from 140m elevation (near monitoring pins), showing high proportion of planktic foraminifera with internal calcite cement and micrite in voids.



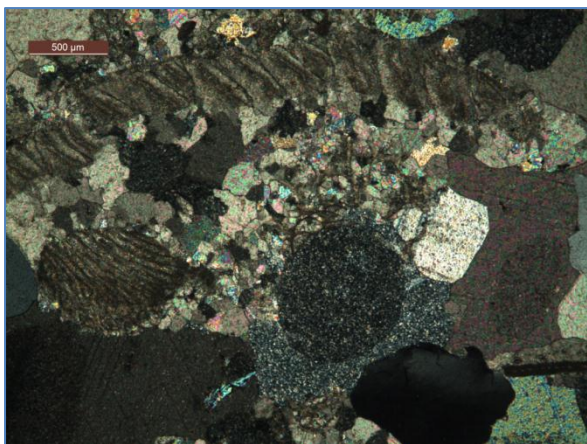
**Figure 6.23:** Thin section of limestone from ~119m elevation at Whitecliffs. Principally composed of benthic foraminifera suggesting a water depth of about 200m.



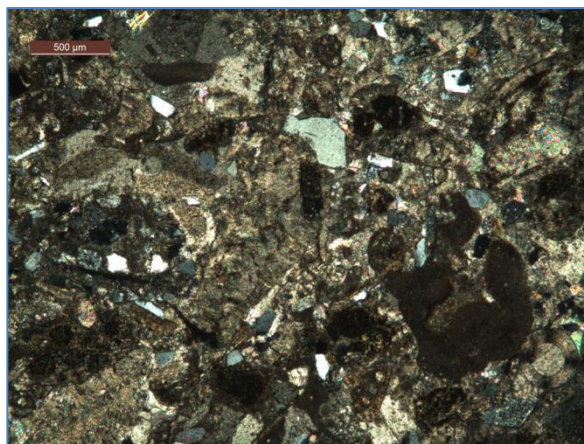
**Figure 6.24:** Thin section of limestone from ~119m elevation at Whitecliffs, showing coarser nature of fossiliferous material and angularity of quartz grains.



**Figure 6.25:** Thin section of limestone from ~118m elevation at Whitecliffs. Planktic foraminifera dominant, with some deep-water molluscs.



**Figure 6.26:** Thin section of limestone from base of cliff at top of talus apron (Profile W2). Shallow water limestone from 50-100m depth.



**Figure 6.27:** Thin section from large displaced limestone block shown on Figure 6.2 at Whitecliffs.

Specific comments are as follows, with discussion from the top of the traverse down for consistency with the descriptions in Section 6.2:

- The sample from 140m amsl, in the vicinity of the monitoring pins, showed a high proportion of planktic foraminifera and some benthic varieties, suggesting a water depth of 200-500m. Some bryozoa and echinoderm fragments are also present (Figure 6.22), but none of the coralline algae associated with shallow water depths lower in the profile. A micritic mud infills voids between the fossil fragments and detrital grains, the latter comprising both biotite and quartz but totalling <2% of the sample. The rock termed a micritic limestone.
- The sample from 119m contains both benthic and planktic foraminifera (Figure 6.23), indicating a water depth of about 200m. The sample contains detrital quartz and some biotite, but with little calcite cement, and the voids between grains and fragments of fossils are infilled by micritic mud. There appears to have been a rapid shallowing in water depth between this sample and the one from 300m further down the profile (Figure 6.24), indicating a possible unconformity, tectonic event or depositional break.
- The sample from 118m contains mostly planktic foraminifera, together with molluscs of deep-water affinity suggesting deposition at the edge of the shelf. A water depth of 200m to 500m is inferred, and again there is a dominance of micritic mud rather than calcite cement. Some very small fragments of detrital biotite are present (<1% of total), and the limestone is can be termed a 'wackestone' (Figure 6.25).
- The sample from the top of the rock talus apron (~75m amsl) is a shallow-water limestone from an inferred depth of 50m to 100m. There is little micritic mud, and instead a well-developed calcite cement. Fossil fragments comprise mostly bryozoa and some coralline algae, with *in situ* glauconite grains growing within fossil fragments (Figure 6.26). Field observations indicated some cross-bedding within this part of the sequence.

In terms of rock mass stability and geotechnical evaluation for rail management, the textures of the limestone samples discussed above are indicative of a strong and moderately indurated limestone, with limited calcite cement and significant micritic mud infilling voids. These observations are consistent with a minimum unconfined compressive strength (UCS) of 40MPa from both core testing and point-load data (density ~2500kg/m<sup>3</sup> and porosity in the order of 5%).

Given the sub-horizontal bedding, which dips back into the face, and the absence of solutioning, long-term stability is inferred based on the intact rock strength. Only fracture-controlled flow of infiltrating water can be expected, and the few open fractures present ensure free drainage preventing pore pressure build-up. It is appreciated that the large (~20,000m<sup>3</sup>) displaced block at SNL97.6km proves that large-scale block failures are possible, and textural analysis (Figure 6.26) suggests that it has been sourced from a mid to upper slope location.

## 6.4 Slope stability assessment

This section presents a qualitative assessment of slope stability for the area defined on Figure 6.1 at Whitecliffs. Slope failure mechanisms and triggering events are discussed in Sections 6.4.1 and 6.4.2, followed by future management considerations, including the speed restriction zone (Section 6.5).

### 6.4.1 Slope failure mechanisms

The mechanisms of failure that are applicable to Whitecliffs are rockfalls and/or topples. The landslide inventory developed in Chapter 3 (Table 3.5) identified four events, considered to represent rockfalls, between SNL97 and 98km. Two of these were listed as very small ( $<10\text{m}^3$ ) to small volumes ( $<100\text{m}^3$ ) at SNL97.50 and 98.00km, based on information from KiwiRail and aerial photograph interpretation respectively.

Duckworth (1968) and Fitzpatrick (2007) both refer to rockfalls as a result of the 1968 Inangahua Earthquake, but no precise locations or volume estimates were provided. The absence of any reported large volume rockfalls since the rail was completed at this end of the Lower Buller Gorge in the 1940s, and the aerial photograph interpretation, indicates that the cliff has been relatively stable for at least the last 70 years. This is supported by the following points:

- The rail corridor at the eastern end of the Lower Buller Gorge only opened in 1942. Construction of the line would have required clearance of a section of the existing rock talus apron by blasting, or other methods, including around the large displaced block adjacent to SNL97.6km (Figure 6.17).
- The limestone is generally flat-lying but does dip back into the face (north to northwest) at a shallow angle (measured between  $5$  and  $14^\circ$ , refer Table 6.2). In comparison, the limestone outcrop adjacent to SH6 is dipping out of the face, which resulted in large-scale block failures in 1968 due to seismic shaking.
- The existing steep and blocky rock talus apron and vegetation cover at the base of Whitecliffs reduces the potential for rockfalls from the cliff-face to reach the rail corridor.
- No limestone blocks greater than  $3\text{m}^3$  in volume, which were not covered by lichen or moss, were observed during inspections of the talus apron between 2009 and 2011. The only fresh rockfalls observed were at a remote distance from the rail, including around the eastern side of the large displaced block (total volume  $\sim 2\text{m}^3$  but individual blocks size only up to  $0.5\text{m}^3$ ), and on the naturally formed ledge at the base of the cliffs/top of the talus apron ( $<1\text{m}^3$  volumes).
- Fractures 1, 2 and 3 mapped at the top of Whitecliffs represent widely-spaced joint sets ( $>10\text{m}$  spacing, but slightly closer ( $\sim 5\text{m}$ ) between the western extent of Fracture 1 and Fracture 2).
- No measureable offsets have been recorded during the monitoring of selected detached limestone blocks by KiwiRail at the top of Whitecliffs (first round completed in 1993, refer Appendix 1.3).



- There were no bedding plane shear zones observed during the descents of Profiles W1 and W2, and only two thin (~300mm thick) mudstone layers observed that could be prone to long-term slaking.
- Analytical testing outlined in Section 6.3 indicates low porosity, high density and associated high intact rock strength that are applicable to the cliff-face profile.
- The photographic record of the cliff-face profile provided by KiwiRail in the 1970s and 1980s does not show any visible changes to the present day (Chapter 3, Figures 3.1 and 3.2).
- Aerial photograph interpretation has not identified any changes in the cliff-face profile between 1947 (earliest photograph) and present day, including a review of photographs flown in 1970 approximately 15 months after the 1968 Inangahua Earthquake (Figure 6.28).

In the absence of details regarding previous failures, the following slope stability assessment considers both of the dominant triggering mechanisms (rainfall and seismicity) in the Lower Buller Gorge, and the future susceptibility for rockfalls/topples.

#### **6.4.2 Rainfall-induced rockfalls or topples**

There have been no reported rainfall-induced landslides from bedrock sources at Whitecliffs to date. In considering the susceptibility of the limestone at Whitecliffs to rainfall-induced failures, Hoek (2007) lists the following triggering factors that may cause a slope to fail as a result of changes in the forces acting on a rock outcrop:

- Pore water pressure increase (rainfall infiltration)
- Rainfall-induced erosion of surrounding material
- Freeze-thaw processes
- Chemical degradation or weathering
- Root growth or leverage by roots moving in high winds

Pore water pressure increases are relevant to how a rock outcrop behaves but discontinuities present within rock masses prevent excessive water pressures during high intensity rainfall events as they are predominantly free draining slopes (Pantelidis, 2009). This factor is directly applicable to the limestone outcrop at Whitecliffs where the joints and fractures in the rock mass preclude excessive pore water pressures during high intensity or long duration rainfall events, and the low porosity of the intact rock prevents significant infiltration even under prolonged rainstorm conditions. Despite the free draining nature of many rock masses, Pantelidis (2009) does list the following five points in regards to how the presence of water can influence slope stability:

- Water pressure reduces slope stability by reducing the shear strength of potential sliding surfaces.
- Water pressure in tension cracks, or near vertical fissures, reduces slope stability due to an increase in the sliding force.
- Erosion of discontinuity infilling can result in a reduction in rock mass stability and also lead to sedimentation of other drainage paths.
- Expansion of freezing water within discontinuities can result in degradation of the rock mass due to large diurnal temperature changes.
- Freezing of surface water on slopes may block drainage paths resulting in the build-up of water pressure in the slope with a consequent decrease in stability.

Potential sliding surfaces were not identified during the two abseiling descents (Profiles W1 and W2), and bedding was observed and measured as dipping back into the face at low angles (refer Table 6.2). This suggests that water pressure has very limited influence on reducing slope stability. It is recognised that a complex sliding surface may be present made up of discontinuities and fractures through the intact rock (Wyllie and Mah, 2004).

Erosion of discontinuity infilling is also not considered to influence slope stability at Whitecliffs as there were little or no fine material noted in the open fractures during abseiling. The infilling of Fractures 1 to 3 at the top of Whitecliffs is also not experiencing erosion. Freeze-thaw processes are not a major concern as the typically moderate temperatures experienced on the West Coast preclude the presence of snow or ice in winter months. Moderate frosts are common during winter months, but are not considered as an influencing factor for slope stability at Whitecliffs.

The main influence rainfall has at Whitecliffs is chemical degradation and weathering of the exposed surface. The type of failure due to this process is shown in Figures 6.15 and 6.16, comprising small volume ( $<10\text{m}^3$ ) blocks being released periodically over time. These blocks are not likely to reach the rail corridor as the talus apron is currently acting as a detention area.

Based on the site model developed, and an understanding of the influence rainfall has at Whitecliffs, it is concluded that this potential triggering mechanism is not of concern for the future stability of the near vertical limestone cliffs. A large seismic trigger ( $>M6$ ) with a shallow hypocentre in close proximity to Whitecliffs (e.g. within a 10km radius) is considered to be the most likely scenario that could induce a failure of a scale to impact rail operations. This is discussed further in the following section.

### 6.4.3 Earthquake-generated rockfalls or topples

Earthquake-generated landslides, including rockfalls, have been reported in the Lower Buller Gorge, as summarised in Chapter 3 (Table 3.5). The majority of these are related to the 1968 Inangahua Earthquake but reference was also made in Chapter 3, Table 3.5 to a ‘major slip’ and minor rockfalls adjacent to SH6 after the 1991 Westport Earthquakes (M6.0, 6.1 and 5.8).

The susceptibility of Whitecliffs to seismic shaking has already been tested during the 1968 Inangahua Earthquake. Both Duckworth (1968) and Fitzpatrick (2007) refer to slope movement at Whitecliffs due to the earthquake but no details on volume or a precise location were recorded. The aerial photograph flown in 1970 that covers Whitecliffs is reproduced from Appendix 3.4, which supports the conclusion that there were no large volume rockfalls or topples at Whitecliffs due to seismic shaking in 1968, either onto or immediately above the rail corridor (Figure 6.28). Earthquake-generated landslides are evident in the 1970 aerial photographs originating on slopes to the north of the rail corridor (circled on Figure 6.28) and many are visible adjacent to SH6.

In the absence of a large ( $>1,000\text{m}^3$ ) failure at this time, impacts on Whitecliffs during the 1968 Inangahua Earthquake appear limited to dilation of existing vertical to sub-vertical fracture sets. The reference made by Duckworth (1968) to the fact that these fractures appeared due to the earthquake are not considered substantiated. Impacts from the 1929 Buller Earthquake have not been identified since the rail was not constructed at the eastern end of the gorge at the time it occurred.

Despite the absence of notable failures at Whitecliffs (adjacent to the railway) in 1968, the region is seismically active (refer Figure 2.6, Chapter 2). It is understood that structural inspections are conducted in the event of an earthquake, as occurred recently in Christchurch and surrounding areas. These inspections will also determine whether landsliding has occurred. Fault sources identified in the region are outlined in Table 5.5 in Chapter 5, based on research by Stafford (2006). It was also concluded by Stafford (2006) that the historic record of observed seismicity in the Buller and Northwest Nelson region can be used for modelling future earthquake occurrence, and that previous research in this area has overestimated the degree of uncharacteristically high seismic rates.

The block of limestone forward to the outside of Fracture 3 has the following approximate dimensions, based on Figures 6.4 and 6.17:

- Plan area:  $30\text{m}^2$  (triangular shape)
- Trace length: 12m
- Depth: 35m maximum (between 140 and 105m amsl)



**Figure 6.28: 1970 aerial photograph in the vicinity of Whitecliffs and SNL97km to 99km (supplied by Aerial Surveys Limited).**

In the absence of more precise survey data, a volume of  $1,000\text{m}^3 \pm 200\text{m}^3$  is estimated. This quantity could be dislodged by high horizontal acceleration during a major earthquake, with toppling onto the rock talus apron being geotechnically feasible. There is no evidence to suggest that failures in this order of volume occurred during the 1968 Inangahua Earthquake, including the absence of such failures in photographs taken in the 1970s (Chapter 3, Figures 3.1 and 3.2). Failure of the slope back to Fracture 1 or Fracture 2 is not considered feasible based on the assessment made in this thesis.

## 6.5 Future management considerations and speed restriction zone

The recommended approach for future management of the rail corridor at Whitecliffs, including the 25km/hour speed restriction, is based on the research findings presented in this thesis. The recommendations outlined are provided for KiwiRail to consider in conjunction with any risk tolerance or judgement values specific to their operations in terms of landslide risk management.

One of initiating factors for conducting research in the project area was to review the Whitecliffs speed restriction. Train speeds within rail corridors may be restricted for a number of reasons, including settlement or deflection of the track formation leading to possible derailments, and temporary restrictions are at time required in the Lower Buller Gorge due to localised embankment instability. The permanent speed restriction of 25km/hour at Whitecliffs is considered as based on a perceived risk. No documentation or knowledge regarding the implementation of the restriction has been identified to date. There is no indication of an imminent failure at Whitecliffs that necessitates a speed restriction, particularly for the very short length of track specified by KiwiRail. The present slope stability assessment of at Whitecliffs has concluded that there is no evidence for active slope movements, with only the potential for small ( $\leq 100\text{m}^3$ ) block failures reaching the talus apron, but it is accepted that on a longer timeframe (100-1,000+ years) large slope movements are possible due to natural weathering or earthquake-triggering events.

A seismic trigger exceeding that of the 1968 Inangahua Earthquake is expected to be the only cause for failure of the limestone cliff of sufficient volume to impact rail operations in the short-term. Track inspection protocols are assumed already in place in the event of seismic activity. Based on the New Zealand research conducted by Hancox et al (2002), a magnitude 5 or greater earthquake is considered a minimum threshold for generating landslides. This level (M5) is considered a conservative threshold for Whitecliffs but other areas in the Lower Buller Gorge will be more susceptible to seismic shaking, particularly if the earthquake occurred during a period of prolonged or intense rainfall. Whitecliffs is not as susceptible to failure as other slopes inspected in the project area due to the structural controls, primarily being the dipping of strata back into the cliff-face and widely space joint sets.

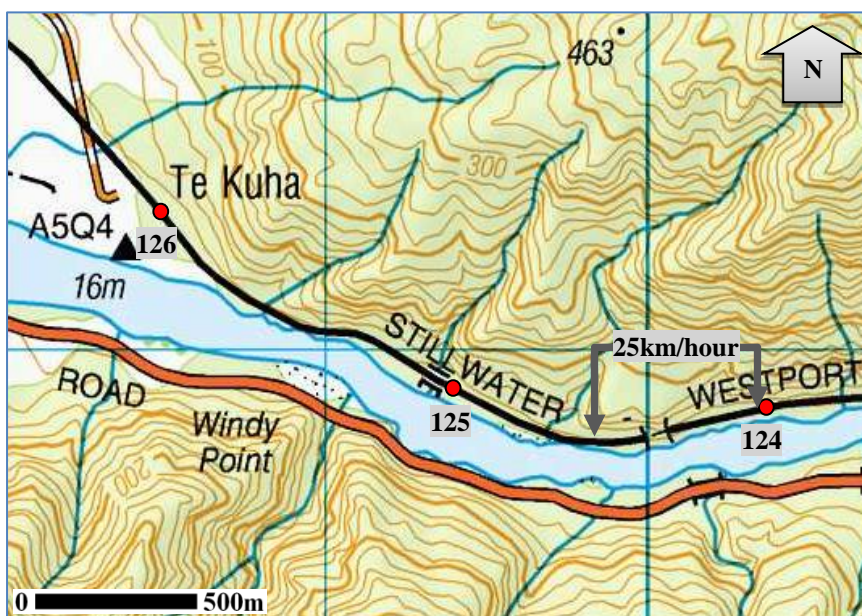
In response to the future risk of earthquake-generated rockfalls or topples at Whitecliffs, it would be possible to set up a real-time telemetered early warning system utilising extensometers. The  $\sim 20,000\text{m}^3$  limestone block at SNL97.6km indicates that large-scale failures have occurred prior to rail construction. A failure of this magnitude would cause significant impacts to rail operations in terms of time and economics. There is no feasible remedial measure available to reduce the impacts from a failure of this magnitude, but an early warning system would assist the continuation of safe operations.



## CHAPTER 7: TE KUHA

### 7.1 Introduction

The area referred to as Te Kuha in Chapter 7 is located at the western extent of the Lower Buller Gorge. As seen in Figure 7.1, the topography above the rail corridor is steep (typically  $\geq 35^\circ$ ), with three main watercourses, originating in steep catchments, intersecting the rail corridor. The 25km/hour speed restriction at Te Kuha currently extends from SNL124.05 to 124.50km. This 450m length of the rail corridor is indicated on Figure 7.1. It is understood from discussions with rail personnel that the rationale and timing behind the speed restriction is not known with certainty, as outlined in Chapter 3 (Section 3.3.2).



**Figure 7.1: Location of Te Kuha and speed restriction (25km/hour) between SNL124.05 and 124.50km.**

The purpose of Chapter 7 is to present a site model for the Te Kuha section of the rail corridor between SNL124 and 126km. Landslide hazard mapping and rock mass characterisation was conducted during numerous site inspections between 2009 and 2011 within this area. The site model presented in this chapter can be adopted by KiwiRail to determine risk tolerance and judgement criteria in accordance with the Landslide Risk Management Framework adopted by Fell et al (2008a).

### 7.2 Corridor geology

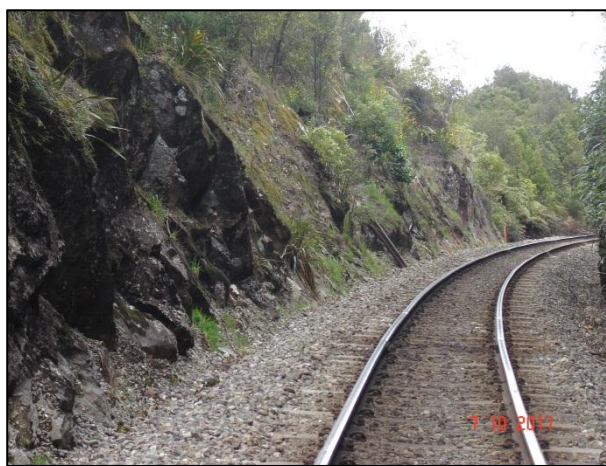
Published geological maps presented in Chapter 2 (Section 2.4) describe the lithologies present between SNL124 and 126km as:

- Tuhua Granite: Undifferentiated, massive or porphyritic, white, grey or pink potash-alkali or calc-alkali granite banded gneiss, adamellite, granodiorite and diorite (Bowen, 1964).
- Rahu Suite: Biotite granodiorite and tonalite (Nathan et al, 2002).

In addition, Nathan (1978) describes these units as ‘*Black biotite quartz diorite, locally foliated; areas containing abundant pink potash feldspar (introduced metasomatically)*’. It is accepted that granitic rocks are the predominant lithological unit in the Te Kuha area, but field mapping also identified areas of metamorphosed Greenland Group inclusions within the younger granites. An outcrop adjacent to the eastern portal of Tunnel 6 at SNL124.208km shows the two distinct units (Figure 7.2).



**Figure 7.2: Outcrop at SNL124.208km (eastern portal of Tunnel 6) showing granite overlying weakly foliated Greenland Group rock. Contact is arrowed.**



**Figure 7.3: Box cutting near SNL125.50km in metamorphosed Greenland Group bedrock.**

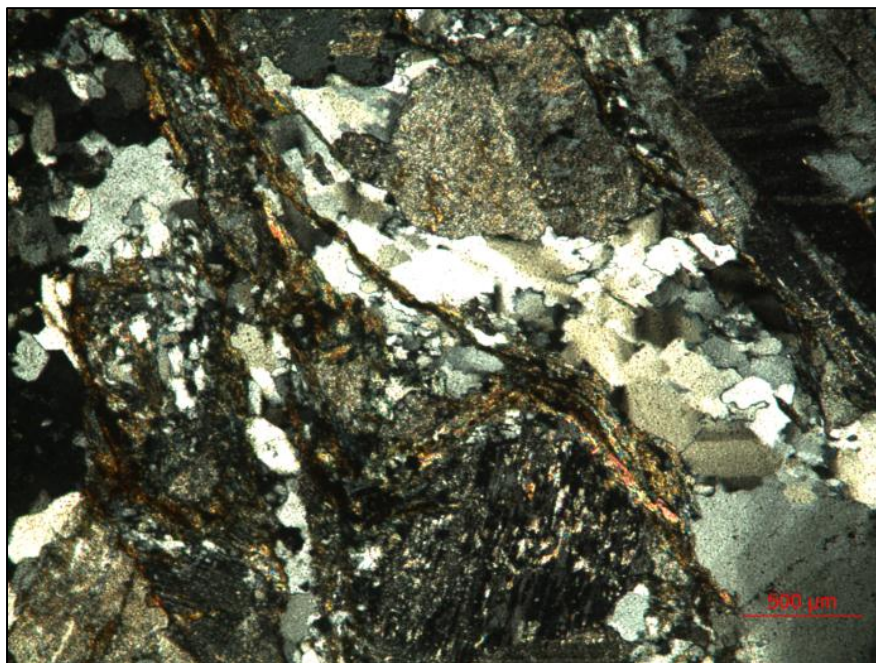
Based on the observations made at Te Kuha, xenoliths of Greenland Group have been incorporated into the granite at a number of locations and show significant recrystallisation to a mica-schist. The main exposure occurs within a 70m long box cutting between SNL124.470 and 125.540km, as shown in Figure 7.3.

Fill material is present around SNL124.578km (78m west of the speed restriction area), and was used to form an embankment. The hillside is not located adjacent to the rail in this area and is not considered susceptible to landslides originating from upslope sources (as far as SNL125km).



### 7.2.1 Thin sections

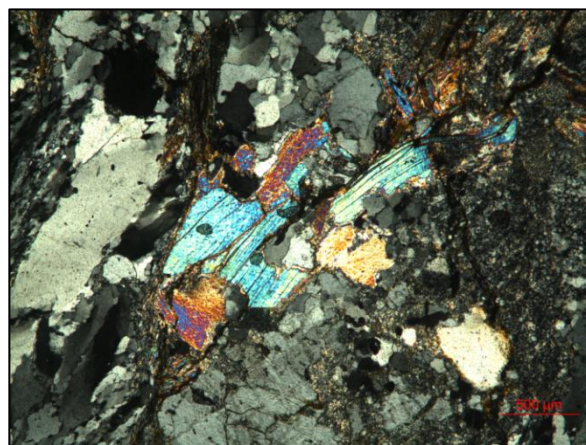
Thin sections were prepared for a number of samples collected along the rail corridor at Te Kuha. Figures 7.4 to 7.8 show lithological details at a scale of 500µm from: SNL124.00km; 124.50km; 125.00km; 125.54km; and 125.63km respectively.



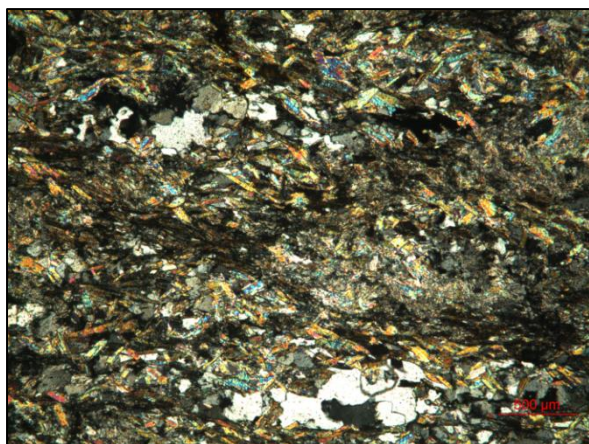
**Figure 7.4:** Thin section of slightly weathered granite from SNL124km showing quartz, mica and sericite-altered orthoclase feldspar. Note low porosity, interlocking texture, and general absence of weathering consistent with equivalent UCS strength >50MPa from Schmidt Hammer and point-load testing.



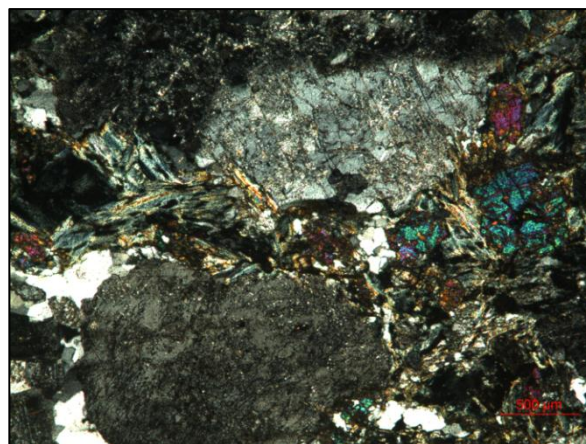
**Figure 7.5:** Thin section of slightly weathered schistose Greenland Group sample from cutting at SNL124.5km. Note interlocking texture of quartz, biotite and slightly altered feldspar, and finer crystal size compared to sample of similar origin from SNL125.54km in Figure 7.7.



**Figure 7.6:** Thin section of slightly weathered granite from SNL125km showing interlocking quartz, mica and altered feldspar. Texture similar to sample from SNL124km in Figure 7.4.



**Figure 7.7:** Thin section of slightly weathered schistose Greenland Group sample from SNL125.54km. Note interlocking texture and low porosity of foliated rock material containing quartz, biotite and slightly altered feldspar.



**Figure 7.8:** Thin section of slightly weathered granite from SNL125.634km showing interlocking quartz, mica and altered feldspar. Texture similar to granite samples from SNL124km and 125km (Figures 7.4 and 7.6).

### 7.3 Landslide occurrence and susceptibility

Landslide susceptibility mapping categorised the length of rail between SNL124 and 126km as ‘high’ (Chapter 4, Figure 4.4) based on steep topography, the presence of surficial colluvium/regolith overlying slightly weathered granite, and recorded landslide events. The landslide inventory developed in Chapter 3 shows the following recorded events between SNL124 and 126 (listed from east to west), with modifications and comments where appropriate from site observations made in October 2011:

- 124.00: small to medium volume landslide (track closed) reported by KiwiRail, triggered by rainfall on 30 July 2008. Observations at this location did not indicate a feature considered to comprise a medium volume (100-1,000m<sup>3</sup>) landslide, but a feature near SNL124.16km was mapped as comprising ~150m<sup>3</sup> of material released (Figure 7.15, Section 7.6). This is considered to represent the landslide referred to by KiwiRail as occurring in July 2008.
- 124.27: large volume landslide reported by PGL (2007), triggered by rainfall. This event correlates to the landslide observed near the western portal of Tunnel 6, and is discussed in Section 7.3.2.
- 124.40: medium volume landslide observed in aerial photography flown in 2000. Observations made at this location are presented in Section 7.3.2, and these have determined this event should be reclassified as ‘large to very large’.
- 125.00: large volume landslide in January 1987 triggered by rainfall. This event has to refer to the Windy Point landslide (SNL125.40km), as presented in Section 7.3.1 (see also below).

- 125.40: large volume landslide reported by PGL (2007). This location also refers to the Windy Point landslide (Figure 7.9), and is considered a more accurate location based on site observations.
- 125.50: train struck small volume landslide on 6 March 2011. This location is within the box cutting shown in Figure 7.3 and based on observations made on site is considered to be within the very small volume category ( $<10\text{m}^3$ ).
- 125.80: unknown volume landslide reported by KiwiRail on 23 April 2011. Landslide observations made in October 2011 determined this feature to be within the small volume category ( $\sim 80\text{m}^3$ ), as shown on Figure 7.16 (Section 7.6). The location of the landslide is considered closer to SNL125.70km.

While it is recognised that the examples listed above do not capture every landslide event, since the rail at this end of the corridor became operational in the mid-1920s, it does highlight that slope stability is an on-going operational challenge to KiwiRail, with at least two occurrences in 2011 alone.

The close proximity of the rail to the adjacent hillslope has been identified throughout this thesis as a contributing factor to landslide susceptibility and impacts. The travel distance component of the landsliding process does influence the ability of material from remote (upslope) sources to reach the rail alignment, but the majority of landslides observed have occurred in material from slopes immediately adjacent to the track formation. Measurements made of the distance from the edge of rail to the adjacent slope are summarised below for representative locations between SNL124 and 126km to illustrate the range present, and proximity:

- 124.000: 1.6m
- 124.451: 8.0m
- 124.500: 5.0m
- 124.528: 2.2m
- 124.578: 3.0m
- 125.000: 2.7m
- 125.020: 2.2m
- 125.146: 5.0m
- 125.634: 1.9m

Within the box cutting (Figure 7.3), the distance from the rail to edge of the cut was consistently around 1.8 to 1.9m on both sides of the track between SNL124.470 and 125.540km. Joint-controlled slab failures in this 70m long cut are not expected to exceed  $10\text{m}^3$  in volume due to the high intact rock strength, and the fact that only localised wedge failures have been identified. The presence of schistose Greenland Group meta-sediments in the cutting is not regarded as significant in terms of rock mass stability, as the foliation is not causing large ( $> 2\text{m}^3$ ) block release. Root penetration into joints can increase the slope susceptibility to small-scale failures over time.

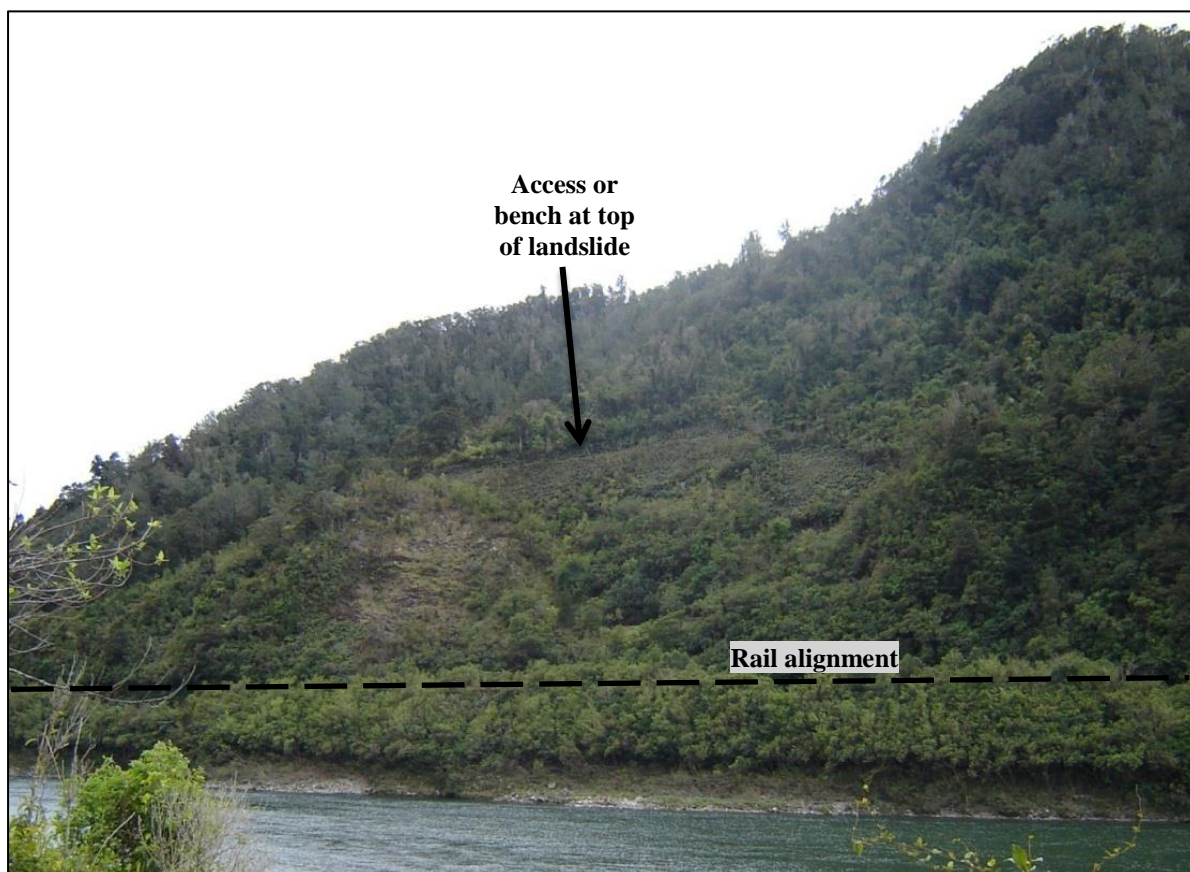


It is recognised that the cost of increasing the catch berm within the Te Kuha section is an expensive option, but due to the proximity of the cut slopes to rail it does not take a large volume of material to impact operations in the event of a train striking rock on the tracks. This also applies to all areas within the project area with a short catch berm adjacent to a steep slope.

Site observations made in February 2011, including a photographic record, and interpretation of slope stability for the area between SNL124 and 126km, are provided in the following sections based on actual landslide occurrences. Volume terminology for landslides follows that used in this thesis.

### 7.3.1 Windy Point landslide (SNL125.40km)

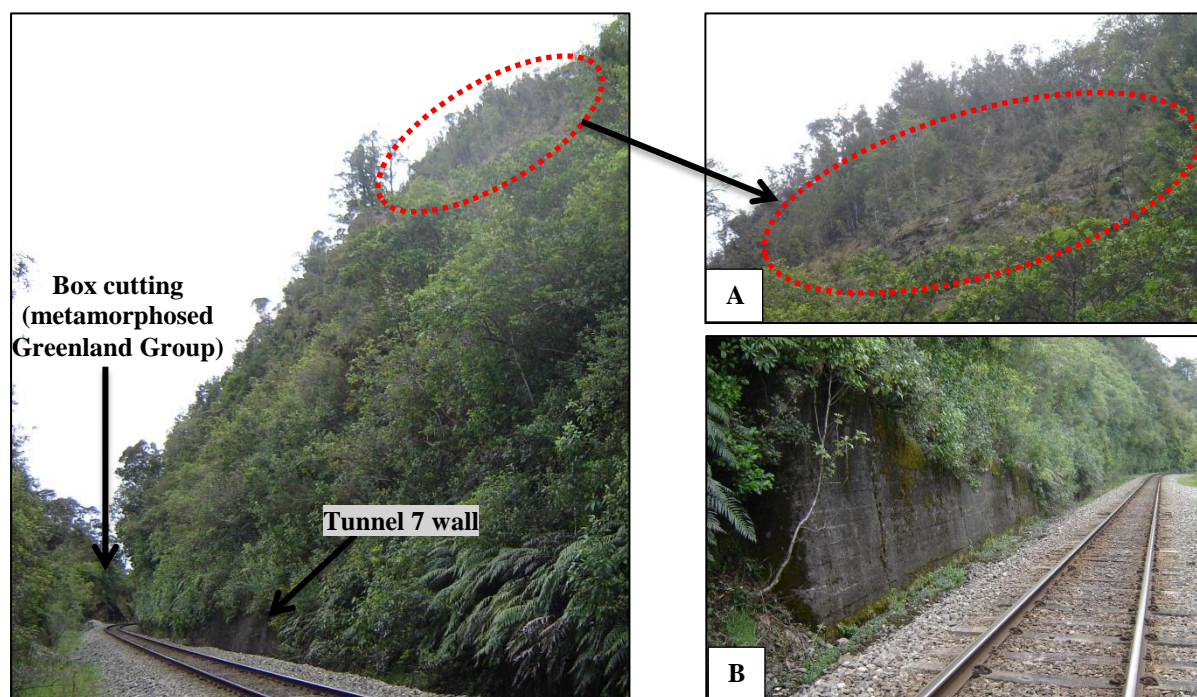
The largest volume landslide documented in the Te Kuha section occurred at Windy Point, around SNL125.4km (~900m west of the speed restriction zone). A photograph of this landslide, Figure 7.9 taken from SH6, shows the extent of the material released. Based on approximate dimensions of 50m (height) x 120m (width adjacent to rail corridor) x 3m average depth, the failure was in the order of 18,000m<sup>3</sup>, placing it within the ‘very large’ category (>10,000m<sup>3</sup>).



**Figure 7.9: View from SH6 of rail alignment and landslide above Windy Point (Tunnel 7) that occurred in January 1987 around SNL125.40km. The tunnel was subsequently daylighted.**

An access or bench across the top of the landslide is apparent in Figure 7.9, as indicated by the arrow. Re-vegetation of the landslide surface area over the last 24 years has effectively stabilised the majority of the slope and there is no evidence of headscarp regression. The decision to daylight Tunnel 7 in 1987 was based on repeated slope failures above the rail corridor between 14-20 January, 28-29 January and 3-4 February 1987, as reported Appendix 3.1. This long-duration rainfall event also caused major disruption to other transportation routes in the region.

Additional views of the Windy Point landslide are provided from rail level in Figure 7.10, including Insets A and B. The box cutting referred to in Figure 7.3 (metamorphosed Greenland Group) is shown on Figure 7.10. Exposed granite bedrock is highlighted in Figure 7.10 and Inset A. The northern wall of Tunnel 7 remains partially intact (Figures 7.10 and Inset B). The original length of Tunnel 7 was ~100m. It is understood that inspection reports from 1987 regarding the Windy Point landslide were prepared (referenced by PGL, 2007), but these were not available for the current project.



**Figure 7.10: View of Windy Point landslide looking west. Inset A shows the exposed bedrock near the top of the landslide, and Inset B the remaining Tunnel 7 wall (view looking east).**

### 7.3.2 Western portal of Tunnel 6

Two large volume landslides were observed near the western portal of Tunnel 6, which is located between SNL124.208 and 124.256km (48m in length). These landslides occur within the current 25km/hour speed restriction zone, as shown on Figure 7.11. The failures are not visible from SH6, but photographs from rail level are provided in Figures 7.12 and 7.13.



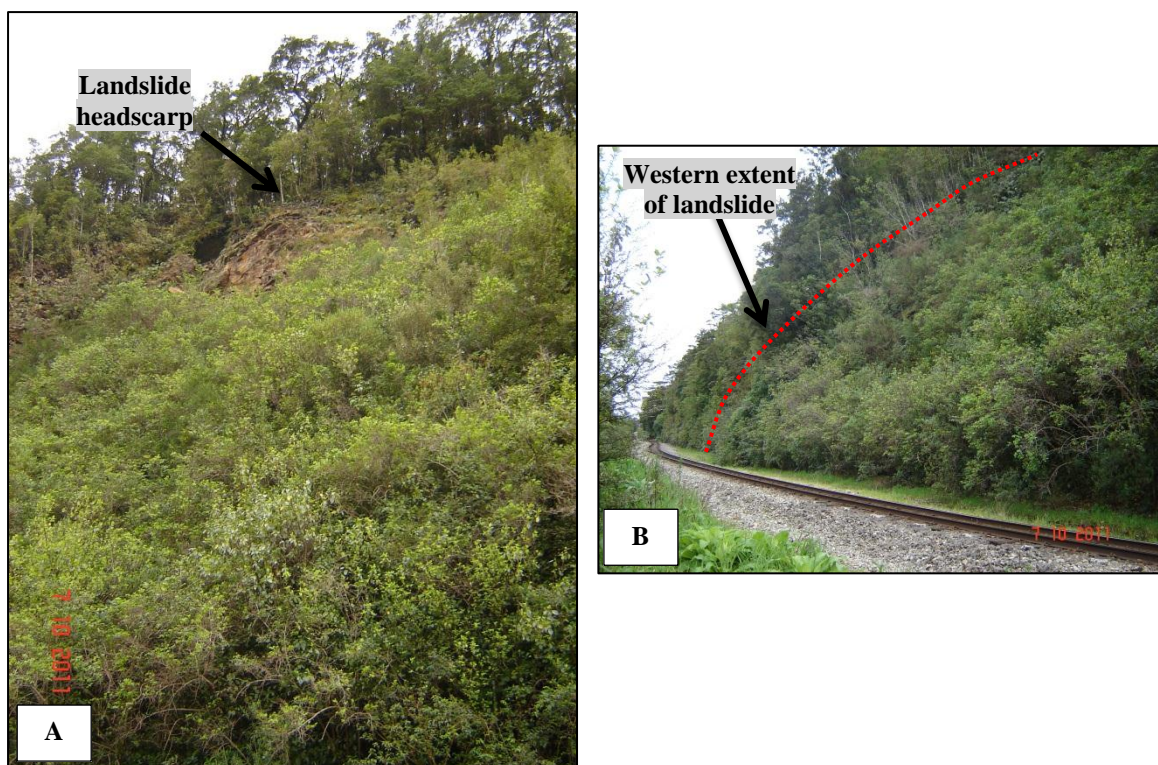


**Figure 7.11: Te Kuha speed restriction area. Slope movements highlighted on the western side of Tunnel 6. Arrows at higher elevations indicate other slope instability features and gullies.**



**Figure 7.12: Views of landslide near the western portal of Tunnel 6 (~SNL124.26km), including the landslide headscarp (A) and close proximity to the rail (B).**





**Figure 7.13: Views of landslide ~100m west of feature shown in Figure 7.12 at ~SNL124.40, including landslide headscarp (A) and western lateral extent (B). Note extensive re-vegetation.**

Both failures near the western portal of Tunnel 6 had similar dimensions: ~50m in height and 80m width adjacent to the rail. The depth of failure is difficult to estimate, but based on an average depth of 2.5m the volume of material released is in the order of 10,000m<sup>3</sup> per landslide. Granite bedrock is exposed in both of the landslides, but re-vegetation has covered most of the failure surface. The difference in tree height at the top of the landslide in Figure 7.13 provides an indication of the age of the features, these being estimated at least 10 years old. This is supported by the aerial photograph in Appendix 3.2 which shows landsliding in this area around 2000.

PGL (2007) noted that the landslide at SNL124.27km has been present in rail operational records since 1995. Sluicing loose debris from the landslide surface was conducted in 2003 to reduce impacts during future long duration or high intensity rainfall events. Deposition of rock material onto the rail in January 2006 was reported by PGL (2007). These observations indicate continuing instability for at least a decade around SNL124.40km.

The failure mechanism for the two landslides near Tunnel 6, and the Windy Point landslide, is translational sliding comprising both colluvium and shallow regolith (weathered granite). This mode of failure is applicable to all areas between SNL124 and 126km that have steep slopes ( $\geq 35^\circ$ ) immediately adjacent to the rail.

## 7.4 Rock mass characteristics

As part of the evaluation of the stability of the rail corridor between SNL124 and 126km, rock mass and material characteristics were defined, including Schmidt Hammer field tests and limited laboratory testing to determine porosity, density and point-load strength of representative rock samples. Results from the field testing conducted are outlined in Sections 7.4.1 and 7.4.2.

### 7.4.1 Intact rock strength from Schmidt Hammer

Intact rock strength was determined using a Schmidt Hammer for comparative purposes only. The instrument used was a Model L, and typically three or five readings were taken in the same location of exposed bedrock at outcrops between SNL124km and 126km. After deleting erroneous low values the Schmidt rebound number (R) was averaged for each particular site, and the “R” values were also converted to equivalent MPa values using Figure 7 of the manual provided for the Model L instrument. It is recognised that absolute values for intact rock strength would require either point load or unconfined compression testing, but this method allowed recognition of variability of exposed bedrock in the field. Results obtained from the Schmidt Hammer testing are summarised in Table 7.1, and a brief interpretation of the data is given below.

**Table 7.1: Intact rock strength indicative values from Schmidt Hammer testing**

Metrage (SNL km)	Rock type	Mean number (R) and number of tests averaged in brackets	Equivalent UCS MPa value
123.929	Granite	31.2 (5)	34
124.000	Granite	48.5 (4)	60
124.027	Granite	39.8 (5)	46
124.037	Granite	32.8 (5)	36
124.105	Granite	44.8 (5)	54
124.123	Granite	42.2 (5)	49
124.163	Granite	42.5 (4)	50
124.208	Greenland Group	36.6 (3)	42
	Granite	46.6 (3)	56
124.451	Granite	39.3 (3)	46
	Granite	39.2 (5)	46



Metrage (SNL km)	Rock type	Mean number (R) and number of tests averaged in brackets	Equivalent UCS MPa value
124.628	Granite	26.2 (4)	27
125.000	Granite	34.0 (5)	38
125.146	Granite	37.6 (5)	43
125.200	Granite	34.6 (5)	39
125.375	Greenland Group	32.8 (4)	36
125.540	Greenland Group	39.8 (5)	46
125.634	Granite	17.6 (5)	15
125.700	Granite	33.2 (5)	37
Note: Schmidt Hammer calibrated for good quality concrete, not weathered rock. Accordingly UCS values present are estimates only.			

Results for the segment of the rail corridor between SNL123.929 and 125.700km, as summarised in Table 7.1, indicate the following:

- For granite the mean Schmidt Rebound (“R”) Numbers ranged between 17.6 and 48.5, which is equivalent to a UCS range of 15 to 60 MPa. Only one “R” value of 19 was below 25.
- The variation in “R” values in the 16 granite test results is attributed to variable degrees of weathering, most being slightly weathered or moderately weathered in hand specimen.
- Inclusions of metamorphosed Greenland Group within the granite gave a range in “R” values from 33 to 40, and were not significantly different from the equivalent granite data.
- The Schmidt Hammer does not allow testing of strength anisotropy, which would be expected in schistose Greenland Group slope exposures, and this is further discussed in Section 7.5.

This analysis demonstrates high intact rock strength for the entire length of the corridor, despite the limitation of not having actual unconfined compressive strength values. This infers that slope failures that do occur are almost certainly defect-controlled, and do not involve failure through intact rock. Field observations indicate a close correlation between root penetration into joints within the rock mass, and general unloading and relaxation typical of steep terrain. Natural slopes examined above the rail corridor in the Te Kuha section are predominantly  $\geq 35^\circ$ .

At SNL125.5km (Figure 7.3) there is a box cutting up to 6m high, and wedge failures are evident on both sides of the track due to defect-controlled instability. There is no evidence for potentially large-scale rock block falls onto, or adjacent to, the track. Assuming minimum intact rock strength of

25MPa, a critical height of slope of 1,000m can be estimated using the approach of Terzaghi (1962). It is therefore the rock mass characteristics, including defect spacing, orientation and persistence that are critical for slope stability.

#### 7.4.2 Defect orientation

Measurements of defects were collected from accessible exposures of granite and metamorphosed Greenland Group outcrops between SNL124km and 126km to determine whether any preferential orientations existed. A total of 56 poles to planes were measured, and after inspection this was treated as a single population. Defect orientation measurements are summarised on Figure 7.14. The Rocscience DIPS programme was used to display the data. General comments applicable to the 2km section of rail are summarised below:

- All measurements represent joint sets, with wedge-type failures being typical.
- Joint surfaces were moderately rough for most measurement sites.
- Joint spacing was consistently observed around 1.0 to 1.5m at or above track level.
- Joint sets were typically penetrative for between 1m and 5m, occasionally greater.
- Tree and other vegetation roots were observed within joints, causing dilation over time.

Figure 7.14 shows three dominant joint sets, as follows, with orientation expressed as dip and dip direction:

- Joint Set 1 – 77/280
- Joint Set 2 – 60/015
- Joint Set 3 – 53/182

The maximum pole concentration of 9.8% relates to Joint Set 3, which strikes sub-parallel to the track (270°T) and dips downslope at about 50°. This is the primary block release surface. Joint Set 2 is a steeply-dipping ( $\geq 75^\circ$ ) defect set striking approximately normal to the track, and Joint Set 1 dips at 60° back into the face, but in a similar strike orientation to Joint Set 3. The clustering of the poles to these three orientations over the 2km length of rail corridor that is being assessed confirms the single population of rock mass defects between SNL124 and 126km.

Rock mass characterisation has confirmed the importance of the dominant defect sets in controlling block release onto, or adjacent to, the rail corridor, with the generally strong intact rock surface acting to release colluvial or regolith materials, together with some loosened joint-controlled rock debris. An estimated ~56% of this 2km rail section is considered vulnerable to future small-large, shallow translational debris slides, and localised flows.

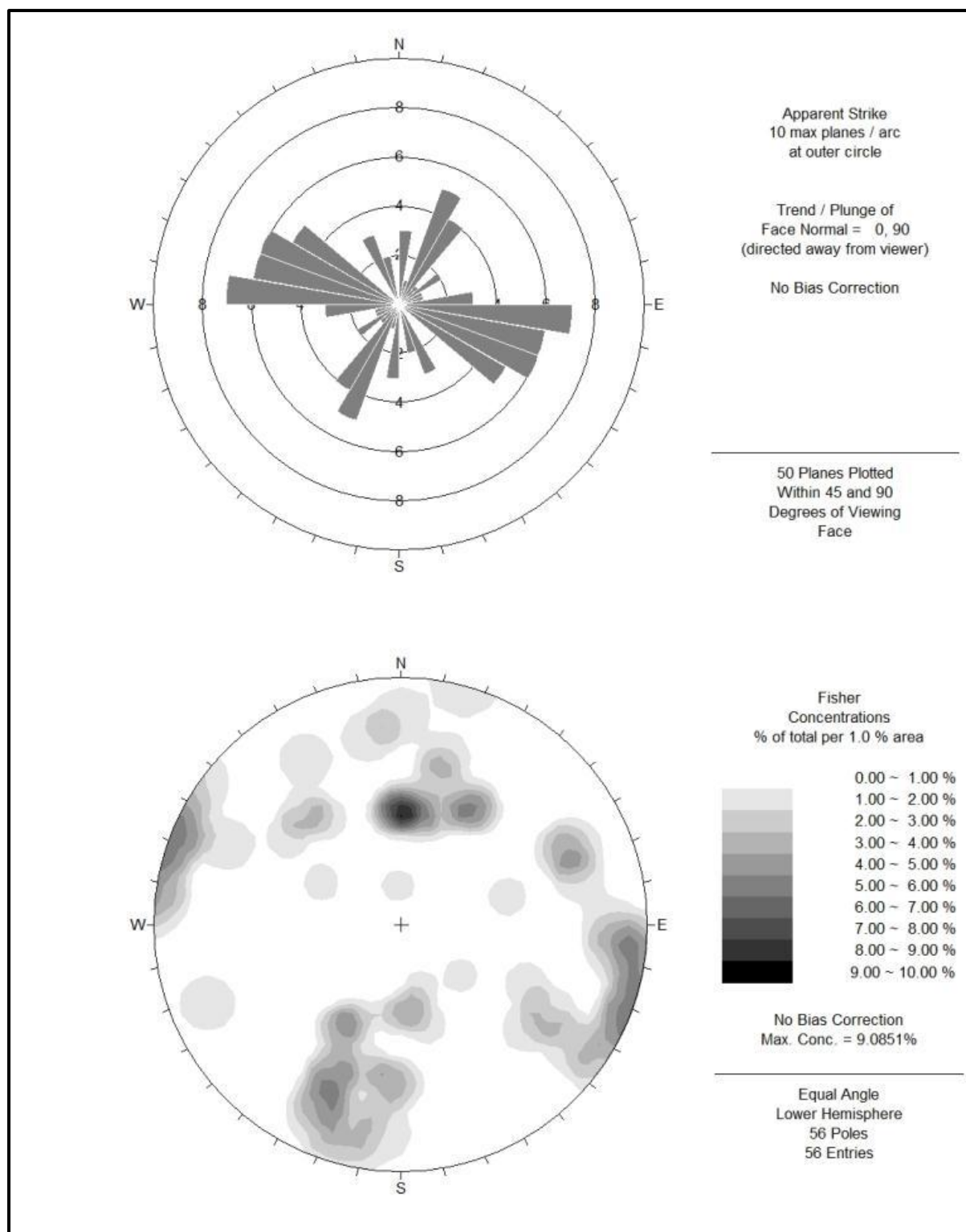


Figure 7.14: Defect orientation (joints) between SNL124 and 126km.

## 7.5 Laboratory testing of rock materials

In conjunction with the field mapping and outcrop Schmidt Hammer testing described in Section 7.4, the following laboratory testing was undertaken on selected samples from rock outcrops adjacent to the rail corridor between SNL124 and 126km, including:

- Granite samples from SNL124km that displayed moderate weathering and some cavities due to partial solutioning of minerals;
- Granite samples from SNL124.163km that were unweathered to slightly weathered;
- Unweathered Greenland Group meta-sediments displaying well developed schistosity from the cutting at SNL125.5km;
- Slightly weathered to unweathered granite from approximately SNL125.65km; and
- Slightly to moderately weathered Greenland Group meta-sediments from within granite at SNL125.77km.

These sites are considered representative of the 2km length of rail corridor, including the 450m length of speed restriction between 124.05 and 124.50km. The *in situ* samples collected comprised rock broken from outcrops or batter exposures. The materials recovered were tested by International Society for Rock Mechanics (ISRM) standard methods for porosity, dry density and point-load strength (Ulusay and Hudson, editors, 2007).

### 7.5.1 Testing results

The data obtained from testing are summarised in Table 7.2, and the mean results for each site are presented. Because of variations in weathering and lithology, and the limited size of samples that could be collected, analysis of physical and mechanical properties was carried out as follows:

- Three samples from each of the five sites listed in Table 7.2 were sawn into cubes, dried and weighed, and then saturated under vacuum. Values for dry density and porosity were computed in accordance with ISRM Standards (Ulusay and Hudson, editors, 2007).
- A total of 21 valid point-load strength values were determined using the irregular lump test (ISRM Standards), and gave limited data on each of the three granite sites (Table 7.2).
- A total of 18 valid point-load strength tests were completed on Greenland Group samples from the two sites outlined in Table 7.2, again following ISRM Standards.

Point-load test for the schistose Greenland Group samples were conducted both normal and parallel to foliation. Data have been analysed in the following sections using the conventional ISRM method of rejecting the highest and lowest values for a single population, and by examining the trends observable for the variation in weathering and specimen anisotropy.

**Table 7.2: Rock materials testing results between SNL124 and 126km**

SNL Metrage (km)	Lithology	Mean porosity (%)	Mean density (kg/m <sup>3</sup> )	I <sub>S(50)</sub> (MPa)	Equivalent UCS (MPa) *
124.00	Granite	5.7 (3)	2,579 (3)	3.89 (8)	93
124.63	Granite	3.5 (3)	2,667 (3)	2.54 (7)	61
125.50	Greenland Group	1.5 (3)	2,839 (3)	9.66 (8)	232 (2) <sup>#</sup>
				3.43 (4)	82 (2) <sup>#</sup>
125.65	Granite	2.9 (3)	2,710 (3)	3.83 (6)	92
125.77	Greenland Group	4.0 (3)	2,742 (3)	4.68 (6)	112 (3) <sup>##</sup>
Notes: Numbers in brackets are numbers of valid tests - * Conversion based on equivalent UCS = 24 x IS(50) - # Schistose Greenland Group inclusions tested normal and parallel to mica foliation - ## Further subdivided on basis of weathering giving 2.5 and 6.9MPa for weathered/fresh samples					

### 7.5.2 Porosity-density data

Results obtained (Table 7.2) reflect the weathering observed, and generally strong nature of the bedrock exposed along the rail corridor. This has been supported by thin-section examination of selected samples (Section 7.2.1), and indicates that the slight variations in porosity (and corresponding density changes) are consistent between the sites. Specific comments are:

- The three granite sites (SNL124.0, 124.163 and 125.65km) show average porosity variations of 2.9 to 5.7%, with corresponding dry densities between 2,710 and 2,579kg/m<sup>3</sup>. These are consistent with weathering grades from unweathered to moderately weathered.
- Samples from SNL124.0km had the highest porosity (4.8 to 6.4%), and displayed the presence of cavities due to solutioning of some unknown mineral (possibility a carbonate). This was consistent with a comparatively lower density range of 2,550 to 2,608kg/m<sup>3</sup>.
- The schistose Greenland Group xenoliths at the Te Kuha site (SNL125.50km) had the highest average density (2,839kg/m<sup>3</sup>) and the lowest porosity (1.5%). The other site at SNL125.77km was more weathered, with lower dry density (2,742 kg/m<sup>3</sup>) and higher porosity (4.0%).

Overall the metamorphosed Greenland Group and granite materials were strong, relatively unweathered, and had low porosities with corresponding dry densities.

### 7.5.3 Laboratory strength data

Insufficient large samples were collected for unconfined compressive strength (UCS) testing, and this was considered to be unnecessary given the high intact rock strength and defect-control of any rock



mass instability. Limited point-load strength testing was undertaken, and average results are summarised in Table 7.2. Specific comments are as follows:

- The point-load strength ( $I_{S(50)}$ ) value for all granite samples tested ranged between 2.54 and 3.89MPa, suggesting an equivalent UCS range of 61 to 93 MPa. Treated as a single population, with the highest and lowest eliminated, the mean  $I_{S(50)}$  value is 3.37MPa. This converts to an equivalent UCS value of 81 MPa using  $UCS = 24 \times I_{S(50)}$  (Bieniawski, 1976).
- Treated as a single population, the mean  $I_{S(50)}$  value for all Greenland Group meta-sediments was 6.44MPa, after eliminating highest and lowest values. This corresponds to an equivalent UCS value of 155MPa using  $UCS = 24 \times I_{S(50)}$ .
- Weathering differences and anisotropy due to biotite foliation were observed in the Greenland Group samples. For the samples from SNL125.5km, the mean  $I_{S(50)}$  values normal and parallel to foliation were 9.66 and 3.43MPa respectively. This indicates an anisotropy index (normal to parallel) of 2.8, and although limited the  $I_{S(50)}$  values are internally consistent.
- Differences in weathering were also evident in the Greenland Group samples tested from SNL125.77km. Although only six were tested, two distinct populations of moderately weathered and unweathered samples were evident having mean  $I_{S(50)}$  values respectively of 2.5 and 6.9MPa. This indicates that the lowest equivalent UCS value is 60MPa for Greenland Group samples in this section of the rail corridor, ignoring any possible foliation-control of instability.

## 7.6 Hazard identification and consequence evaluation

Landslide characterisation and frequency analysis are the main components to consider for the hazard analysis component of the Landslide Risk Management framework outlined in Chapter 1, Figure 1.3 (based on Fell et al, 2008a). Landslide characterisation has been established for the Lower Buller Gorge project area in Chapter 5. Specific comments relevant to the 2km length of rail at Te Kuha based on findings presented in Chapter 7 are:

- The dominant mechanism of failure observed within granitic rocks is translational landsliding of regolith and colluvium over relatively unweathered and strong bedrock;
- Minor topples have been observed from cut slopes, such as the box cutting at SNL125.5km (Figure 7.3), and debris flows may be associated with sediment mobilised in gullies;
- Joint-controlled failures in slightly to moderately weathered bedrock at the regolith interface are not uncommon, but block sizes are typically less than  $10\text{m}^3$  as shown in Figure 7.16;
- The dominant joint sets (1, 2 and 3) are present in many bedrock exposures, and Joint Set 3 (strike sub-parallel to track) is the primary control of bedrock failures by translational sliding;
- Bedrock, where exposed, shows slight to moderate weathering only, and point-load testing indicates equivalent UCS values for granite  $\geq 60\text{MPa}$ ;

- Intact rock strength for the metamorphosed Greenland Group exceeded 100MPa normal to foliation, and there was no field evidence for localised foliation-controlled instability; and
- Previous landslide history, as documented since about 1985, indicates future potential for both large volume ( $>1,000\text{m}^3$ ) and smaller to medium ( $<10$  to  $1,000\text{m}^3$ ) landslides.

AGS (2007) provide a summary table detailing the length of the historical record in terms of estimating return periods (in this case landsliding), and the resultant reliability. The tabulated data was based on research by Lee and Jones (2004), and is reproduced in Table 7.3.

**Table 7.3: Length of historical record required to estimate return period events with 95% and 80% reliability**

Return period (years)	Length of record (years)	
	95% reliable	80% reliable
2.33	40	25
10	90	38
25	105	75
50	110	30
100	115	100

The historical record for this thesis starts from the 1920s, but the very limited information available from this time, up until 2004, could not be considered adequate for frequency analysis. Information provided by KiwiRail was made available from 2004. This data set is considered the most complete, even recognising the absence of technical descriptions or quantification of landslide data in most cases. Based on the seven year record, return periods cannot be reliably calculated for landsliding in the project area, particularly for small to medium volume events ( $<1,000\text{m}^3$ ). According to the guidance provided in Table 7.3, a minimum 25-year record is required for an 80% reliable estimation of 2-3 year return period events, and discussion of frequency in this thesis is therefore subjective.

The frequency of large to very large volume landslides, as described in Sections 7.3.1 and 7.3.2, is expected to be approximately every 10 to 20 years based on the very limited amount of information available over the operational history (approximately 90 years). In comparison, the frequency of small to medium volume landslides is every 2 to 3 years based on the historic record. It is noted that two small volume events were recorded by KiwiRail in 2011 alone, so this frequency can be expected to vary over time in response to climatic conditions. Two examples of small to medium volume landsliding are provided in Figures 7.15 (~SNL124.16km) and Figure 7.16 (~SNL125.70km).



**Figure 7.15: Medium volume (~150m<sup>3</sup>) landslide in granitic regolith observed near SNL124.16km on 7 October 2011.**



**Figure 7.16: Small volume (~80m<sup>3</sup>) landslide in metamorphosed Greenland Group observed on 7 October 2011 near SNL125.70km.**

The Landslide Risk Management framework from AGS (2007) provides an example of factors to consider in terms of consequence analysis. These include elements at risk (property; roads/communications; services; people; distance to the impacted area); temporal probability (for people and/or vehicles in the area); and vulnerability (relative damage; probability of injury/loss of life). The following aspects are relevant to the Lower Buller Gorge project area:

- **Elements at risk:** Includes locomotive engineers, and other rail personnel, infrastructure and rolling stock.
- **Temporal probability:** Considered as the length of time in a specific area, in this case the exposure time of a train within a specific landslide susceptibility zone or defined length of the rail corridor.
- **Vulnerability:** There have been no reported injuries or loss of life as a direct result of landsliding within the Lower Buller Gorge. The vulnerability to injury/loss of life is minimised by low exposure times and the nature of train movements. The principal economic vulnerability is the SNL being relied upon for the transportation of coal to the port of Lyttelton.

Other factors to consider in terms of consequence scenarios as a result of landsliding within the Te Kuha section are:

- **Location:** SNL124 to 126km is located at the western end of the Lower Buller Gorge, closest to the Westport depot and with road access to Te Kuha (Figure 7.1). This means that mobilisation of earthworks machinery for landslide clearance, or emergency vehicles, is comparatively easier than for more remote locations in the project area.
- **Visibility:** Track curvature limits visibility for locomotive engineers through most sections of the Lower Buller Gorge, including between SNL124 and 126km. Visibility is particularly limited within the speed restriction zone (SNL124.05 to 124.50km), and between SNL125.2 and 125.7km. These two areas account for almost 1km of the 2km long Te Kuha section that would require a very short stopping distance (<250m) by trains in the event of encountering rock debris or vegetation on the rail.
- **Catch berms:** As discussed in Section 7.3, due to the close proximity of the steep slopes to the rail alignment (often around 2m), even small to medium volume landslide events have the potential to cause operational delays to KiwiRail.

Large to very large landslides ( $>1,000\text{m}^3$ ) have the potential to block the track for days to weeks, and to cause either derailments (if the train cannot stop in time) or to impact the train as it passes. Such landslides would be expected to be triggered by either severe earthquake shaking, or by prolonged or extreme rainfall. In either case it is assumed that the track would be inspected prior to scheduled train departures to ensure the safety of infrastructure and clearance of any debris that has impacted the rail corridor.

The consequences of large to very large landslides in the Lower Buller Gorge are not considered to pose a major threat due management procedures for track inspections, and the assumption that the rail corridor would be closed immediately for inspection. It is understood that KiwiRail conduct regular ‘wet weather runs’ using high-rail vehicles, but no triggering rainfall level has been specified to initiate an inspection. Inspections are also required in the event of a large magnitude earthquake. Again, there does not appear to be a threshold level set, but the recent seismic activity in Canterbury illustrated the level of inspections undertaken, including structural inspections for all branch lines in the Canterbury and wider region.

Small to medium ( $>10\text{m}^3$  to  $<1,000\text{m}^3$ ) volume events are considered to pose a greater risk in terms of personnel safety, and an increased probability for trains striking debris on the track and possibly derailments. The absence of rainfall thresholds that may initiate landslides in the Lower Buller Gorge means that smaller magnitude events may not necessitate a track inspection, for example if they occur after a period of low intensity rainfall. Difficulties in establishing site-specific rainfall thresholds exist due to the lack of rainfall gauges in the Lower Buller Gorge. Correlating landslide occurrence with more remote rain gauge data (i.e. in Westport or Reefton townships) will not necessarily produce reliable thresholds since rainfall intensity and duration can vary over short distances.

Frequency and consequence analysis is developed further in Section 7.8, which includes a risk calculation example related to trains travelling through SNL124.05 to 124.50 at 25km/hour (current permanent speed restriction) in comparison to 50km/hour, and the probability of encountering debris on the track. It is understood that 50km/hour is the maximum possible speed for laden coal trains travelling through the Lower Buller Gorge due to track curvature and grades.

## 7.7 Te Kuha site model

Details regarding landslide occurrences (Chapter 3, Table 3.5) have been correlated, and updated, based on site observations and mapping conducted in October 2011. The updated summary table of recorded landslides between SNL124 and 126km is presented in Table 7.4. These features are mapped on Figure 7.17, together with infrastructure details (Tunnel 6, Bridge 106 and culverts), topography, the Lower Buller Fault, principal watercourses intersecting the rail corridor, and the 70m long box cutting around SNL125.50km.

**Table 7.4: Updated landslide inventory for occurrences mapped on Figure 7.17 between SNL124 and 126km**

SNL Metrage (km)	Date of occurrence	Volume
124.16	30 July 2008	Medium (~150m <sup>3</sup> )
124.27	First recorded by KiwiRail in 1995	Very large (in the order of 10,000m <sup>3</sup> )
124.40	~2000 based on aerial photograph interpretation	Very large (in the order of 10,000m <sup>3</sup> )
125.40	18 January 1987	Very large (in the order of 18,000m <sup>3</sup> )
125.50	6 March 2011	Very small (<10m <sup>3</sup> )
125.70	23 April 2011	Small (~80m <sup>3</sup> )

Table 7.4 indicates that six landslides are known from the 2km section of track at Te Kuha, although others almost certainly have occurred in the 80+ years since the railway opened from Westport. Of these, three are very large volume events ( $\geq 10,000\text{m}^3$ ), and three are very small to medium ( $\leq 150\text{m}^3$ ).

The three very large landslides are those associated with Tunnel 7 (daylighted in 1987), and the two close to the western portal of Tunnel 6 (which occurred between 1995 and 2000). The three very small to medium volume landslides are indicated by numbered stars on Figure 7.17, and all three have occurred in or since 2008, which suggests others will also have occurred. Areas where landslide susceptibility is considered low, in terms of impacting rail operations, are defined on Figure 7.17, and described below:



- Tunnel 6 (48m in length);
- SNL124.55 to 125.13km due to an absence of source debris and/or a relatively wide clearance on the inside of the track (includes Bridge 106);
- SNL125.75 to 126.00km, where source areas are well removed from the rail corridor.

This suggests a total of ~ 880m (44%) of the Te Kuha section of track has a low susceptibility to landsliding that will impact rail operations, and that the remaining 1,120m (56%) has a moderate to high susceptibility to landsliding onto the track from upslope sources. The implication is that it is difficult to justify the speed restriction zone between SNL124.05 and 124.50km given the comparable landslide susceptibility in the vicinity of the Windy Point landslide (Figure 7.17).

## 7.8 Risk analysis example for the 25km/hour speed restriction zone

The site model developed for Te Kuha shows that within the 450m long 25km/hour speed restriction zone between SNL124.05 and 124.50km (Figure 7.17) there have been at least two large volume landslides (near the western portal of Tunnel 6), and one comparatively more recent (2008) medium volume (~150m<sup>3</sup>) landslide near SNL124.16km. Approximately 200m within this 450m speed restriction zone (or 50% if the 48m tunnel length is taken into consideration) has been impacted by landsliding within the last 15 to 20 years. This high proportion, particularly related to the larger volume events, likely accounts for the decision to impose a permanent speed restriction. The following basic risk analysis illustrates the exposure time variability to landslide occurrence within the 450m speed restriction zone for train velocities of 25km/hour and 50km/hour.

**Scope:** Calculate the risk of encountering a landslide for trains travelling through SNL124.05 to 124.50 at 25km/hour in comparison to 50km/hour.

**Landslide characterisation:** The entire 402m length of rail corridor (excludes the 48m tunnel length) is considered as having the same (high) susceptibility to landsliding based on the consistency of steep slopes ( $\geq 35^\circ$ ) adjacent to the track; geology; and defect orientation.

**Frequency analysis:** The average frequency of very small to medium volumes landslides is considered at 0.5 per annum (or 0.0014/day). This frequency is conservative as it assumes one event every two years, which is more likely applicable to the 2km length of rail between SNL124 and 126km (or more specifically the 1,120m of track considered susceptible to landsliding).

**Consequence analysis:** The temporal probability ( $P_{(S:T)}$ ) of a train occupying the length of rail onto which a landslide could occur is:

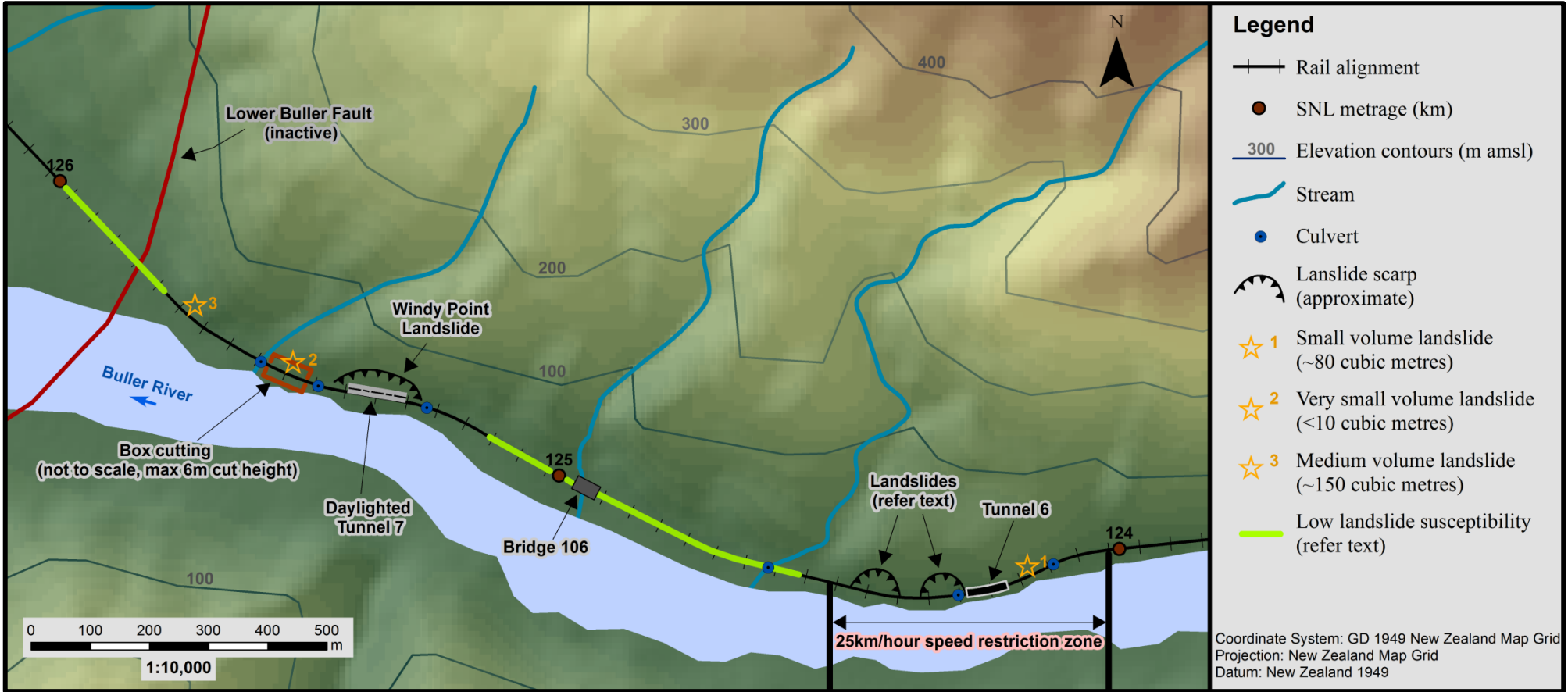


Figure 7.17: Te Kuha site model between SNL124 and 126km showing the 25km/hour speed restriction zone, landslide occurrence, infrastructure features and areas defined as ‘low landslide susceptibility’.

$$P_{(S:T)} = \frac{N_t}{24} \times \frac{L}{450} \times \frac{1}{T_v}$$

Where:  $N_t$  = average number of trains per day

$L$  = average length of train (metres)

$T_v$  = velocity of train (km/hour)

$P_{(S:T)} = 0.011$  at 25km/hour and 0.006 at 50km/hour

The above calculation considers an average of six train movements per day in the Lower Buller Gorge, with each train having an approximate length of 500m (comprising 30 coal wagons and the locomotive).

**Vulnerability:** This is a subjective value applied to the rail corridor itself. An example provided by AGS (2007) indicates a vulnerability value of 0.3 was applied to roads by Michael-Leiba et al (2002), and this value has been adopted for the current analysis.

**Risk estimation:** Considers annual probability of a train being struck by a landslide. Calculated by multiplying: Landslide frequency (0.5/annum) x  $P_{(S:T)}$  x Vulnerability

=  $1.67 \times 10^{-3}$  per annum (25km/hour), and

=  $8.33 \times 10^{-4}$  per annum (50km/hour)

To evaluate the risk, tolerable levels need to be considered. This is a judgement to be made by KiwiRail. If the speed restriction were removed, and potentially doubled to 50km/hour, the exposure time is halved. Inversely, there is a corresponding increase in the stopping distance at full-service brake application for any speed greater than the existing restriction.

Analysis for a train, either laden or empty, hitting debris that has fallen or slid onto the track is not as simple. To determine the risk of striking debris on the track at different speeds, consideration has to be given to all the following aspects (based in part on Barney et al, 2001):

- Visibility distance around the relatively tight curves in the Lower Buller Gorge.
- The delay between sighting something substantial on the track and application of the brakes.
- The brake delay time between the command to the brakes and them becoming effective.
- The deceleration available with full-service brake application once fully implemented.
- The state of wear of the brake-pads and the air pressure available in the cylinders.
- Other factors such as the track geography and the mass distribution of the train.

Barney et al (2001) provide a sound method of calculating train braking distance for Queensland Rail, including development of a PC-based programme. It is cautioned by Barney et al (2001) that calculating train braking distance has a number of uncertainties, and it is considered that a full quantitative analysis of this matter is more appropriately done by KiwiRail using their own specific input parameters now that the landslide hazard has been characterised.

Given that much of the at-risk slope within the restriction zone has now failed, and the debris sources have been largely removed, there seems little justification in continuing with the speed restriction given the increased exposure from slower speeds. It is accepted that the analysis carried out has been based on debris impacting the train from smaller more frequent events ( $<1,000\text{m}^3$  in volume), rather than the larger slope failures which are much less frequent on average (about once every 10-20 years) but which will result from events that are of such a scale that temporary track closure and immediate inspection is appropriate. Analysis of the likelihood of a train being stopped or derailed by sudden smaller debris impacts onto the rail will require specific information on braking distance for the locomotives in use and the nature of the track, including maximum speed, curvature and visibility.

## 7.9 Summary

The rail corridor at Te Kuha is formed at the base of typically steep slopes ( $\geq 35^\circ$ ) in granitic rocks, with inclusions of schistose Greenland Group meta-sediments, and an average 2-3m thick regolith comprising weathered granite blocks with finer gravelly debris. Towards the slope base colluvium may be present and translational sliding at the bedrock interface results in episodic debris slides and occasional flows. Minor rockfalls will occur from steeper bedrock outcrops close to the rail, and joint-controlled block release up to  $\sim 10\text{m}^3$  in volume occurs on a dominant defect set striking sub-parallel to the track and dipping downslope at  $45\text{-}50^\circ$ . Rock strengths are moderately high, typically greater than 50MPa and the intact rock will not fail. Failures will locally be defect-controlled.

Three large volume landslides are documented between SNL124 and 126km. The frequency of small to medium volume landslides are considered to be around one every two years and it is these events that are considered to cause more risk to rail operations due to the absence of rainfall or earthquake triggering threshold values for determining track inspections. It is assumed that wet weather inspections will occur after periods of high intensity or long duration rainfall and/or after a large magnitude earthquake with an epicentre close to the project area.

It is difficult to justify the speed restriction zone between SNL124.05 and 124.50km given the comparable landslide susceptibility in the vicinity of the Windy Point landslide around SNL125.40km. Any speed restriction should be based on the ability for the locomotive driver to stop or adequately slow the train following recognition of the hazard.

## CHAPTER 8: FUTURE LANDSLIDE RISK MANAGEMENT PRACTICES

### 8.1 Introduction

The Landslide Risk Management framework (Chapter 1, Figure 1.3) follows on from analysis to the next stage of risk evaluation and mitigation. As indicated on Figure 1.3, it is considered most appropriate for KiwiRail to establish and evaluate risk tolerance criteria, including value judgements and associated mitigation measures within their existing operational plans.

Chapter 8 is provided as a general guide to the future management of the 30km long rail corridor through the Lower Buller Gorge in relation to landsliding onto the track from upslope sources, based on the research conducted for this thesis. This includes failures originating from local cuttings that were completed as part of railway construction, and debris slides and flows sourced from steep ( $>20^\circ$ ) natural slopes in a number of different lithologies. The structure of Chapter 8 follows the rail management options outlined in Table 8.1.

**Table 8.1: Rail management options for the Lower Buller Gorge relevant to upslope-sourced landsliding**

Category	Methodology
<b>Landslide inventory documentation and analysis</b>	<ul style="list-style-type: none"> <li>▪ Compile existing records, in addition to those recorded in this thesis, noting landslide types, volume, run-out distance, and triggering mechanism (type and magnitude).</li> <li>▪ Use standardised terminology and develop a system or template for recording all relevant data for future landslide events.</li> <li>▪ Identify and analyse sites that experience recurring slope instability.</li> </ul>
<b>Early warning systems and landslide monitoring</b>	<ul style="list-style-type: none"> <li>▪ <b>Rainfall:</b> Establish rainfall triggering levels to initiate rail corridor inspections (Chapter 5).</li> <li>▪ <b>Earthquake:</b> Conduct rail corridor inspections in the event of a close source magnitude 5 or greater event (based on threshold considered conservative but realistic for New Zealand by Hancox et al, 2002).</li> <li>▪ Physical or electrical early warning systems can be adopted, but real-time warning systems are required to minimise the response time for either triggering mechanism.</li> <li>▪ Monitoring of key slope instability sites by KiwiRail personnel, or contractors, familiar with the Lower Buller Gorge environment to determine any changes in slope morphology.</li> </ul>



Category	Methodology
<b>Protection at track level</b>	<ul style="list-style-type: none"> <li>▪ Use of catch ditches or engineered bunds or benches, including vegetation. Would require track widening or realignment in some places and it is recognised that due to access and topographic constraints this will not always be feasible or economically justifiable.</li> <li>▪ This also applies to site-specific design of rock avalanche shelters, or similar, at known recurring landslide locations, including Sinclair Castle and Cascade (Appendix 5.1)</li> </ul>
<b>Slope modification</b>	<ul style="list-style-type: none"> <li>▪ Remove colluvium/regolith at source and/or bench slopes for stability.</li> <li>▪ Implementation of horizontal drains or slot drains to control overland water flow.</li> <li>▪ Use of rock stabilisation measures (e.g. bolting or buttressing).</li> </ul> <p>All of the above options are noted as expensive. The alternative is reactive remedial work in the event of a landslide. To increase efficiency of response it is recommended that dedicated plant (excavators) is available for use at the Westport KiwiRail depot.</p>
<b>Closure of rail corridor</b>	<p>Re-vegetate and abandon rail alignment or convert to recreational use. Requires an alternative means of coal transport (shipping) and is not considered further in this chapter.</p>

## 8.2 Landslide inventory and documentation

The first component recommended in Table 8.1 is the requirement for an updated and live version of a landslide inventory for the purpose of establishing frequency of events of varying volumes in response to rainfall or earthquake triggering mechanisms. It is already recognised that landslide susceptibility does vary through the 30km length of rail between SNL96 and 126km, but in order to quantify the hazard for each occurrence, or section of rail, detail on frequency and consequences requires documentation. This thesis has identified this data gap as the main limitation for detailed landslide hazard analysis.

Recording of landslide events impacting the SNL has improved since 2004, but the absence of robust documentation of landslide events in New Zealand is recognised in other comparable studies. The example provided in this section by Smith (2004) was chosen to illustrate the approach taken in regards to the rail and highway corridors between Arthurs Pass and Greymouth, where it was identified that only limited information on the type of landsliding, volumes and triggering mechanism was available. By combining data from several sources, including Tranzrail Limited and Transfield Limited, Smith (2004) was able to compile a database that could be interrogated or updated. Smith (2004) used the following five major categories in assembling her database, which was termed ‘The West Coast Historical Mass Movement Inventory’:

- **Time:** Included the date of the landslide event, and where there was a sequence of events the first was always identified.
- **Location:** For the Midland Line this included the distance from Christchurch, but these were not always accurate and some confusion arose with changed line names.
- **Mass Movement:** Type of landslide, volume estimate, and triggering mechanism (earthquake, rainfall, unknown). Volumes were frequently the amount of debris removed from the highway or the railway, rather than the actual volume of the slope failure itself.
- **Damage:** Nature of the damage, the period of closure, the number of fatalities (if any), and solutions/comments pertaining to the specific event.
- **Logistical:** This field refers to a unique identified number for the event, the record number, the source of the information, and the availability/inclusion of any photographs.

Smith (2004) incorporated 1072 mass movement events in the database that comprised 805 with a definite rainfall trigger, 139 with a probable rainfall trigger, 80 that were of unknown origin, 35 that were attributable to earthquake, and 13 that were placed in various “other” categories. Railway records spanned a period from around 1920 to 2004. Highway records only began in the 1960s, and the above listing included railway and highway closures or damage. Analysed simplistically, 88% of all entries relate to rainstorm triggering, and 3% to earthquake triggers. The implication of the 80 mass movements of unknown origin (7.5% of the total) is that available records were neither consistent nor detailed enough for a fuller analysis.

It is recommended that a similar inventory system is adopted for the Lower Buller Gorge rail corridor, with information recorded under each of the five main fields listed above. This should be designed as a railway-specific inventory since there are differences between the types and extent of landsliding on each side of the Buller River. In addition, rainstorm triggers are often quite localised, and there are a number of issues (such as a network of rain gauges) that need to be considered. It is important that the system adopted is suited to non-technical personnel doing the recording on a basis and at a frequency that suits the number of train movements and the consequences of derailments.

Once a systematic recording system is established, and refined by trial and error, it will be possible to undertake correlations with rainfall, establish thresholds for inspection or immediate track closure, and put in place a series of protocols to manage the impacts of landsliding affecting the rail corridor. Culvert maintenance, fill batter performance, and sites of erosion potential impacting the formation, could also be incorporated.

### 8.3 Early warning systems

Warning systems can be considered under three broad groupings in regards to the Lower Buller Gorge rail corridor, as follows:

- Rail management protocols that involve temporary track closure and physical inspection by trained staff once certain threshold values of either rainfall intensity or earthquake shaking are reached.
- Physical warning devices or alarms that are triggered and provide immediate advice to rail operators that inspection and/or track clearance is required. Such systems need to be robust, simple to operate and interpret, and not subject to erroneous outputs (Macfarlane et al, 1996).
- Speed restrictions on train movements that are based on hazard recognition, and for which reduced train speed (with increased exposure time) is justified by the threat to operations or personnel. Such speed restrictions presently exist at Whitecliffs and Te Kuha.

Implementation of management protocols based on threshold levels of rainfall or earthquake shaking are both feasible and appropriate for the Lower Buller Gorge. Establishing operational protocols for earthquake response is relatively straightforward, but rainfall triggers do require further research and the establishment of a network of gauging sites as discussed in Chapter 5.

In terms of developing models for specific warning or alarm systems, the most important factor to consider is that the output from any form of modelling is only as robust as the input data. Capparelli and Versace (2010) itemise the following components as critical for an effective early warning system related to slope movements:

- Landslide susceptibility maps;
- Impacts (scenarios) from events;
- Monitoring data for key parameters, including real-time data transmission;
- Mathematical modelling (for current hazard evaluation and future forecasting);
- Warning models;
- Emergency plans; and
- Established decision-making procedures.

While a landslide inventory is not specified in the above list, the reference to landslide susceptibility maps infers that an inventory exists. Impacts or scenarios listed by Capparelli and Versace (2010) are the equivalent of consequence analysis within the Landslide Risk Management framework (Fell et al, 2008a).

In terms of mathematical modelling, examples were provided by Capparelli and Versace (2010) that were developed for assisting in early warning of rainfall-induced landslides. The two models, developed by the authors, both consider triggering factors. The titles and summary of each are detailed below:

- **Forecasting of Landslides Induced by Rainfall (FLaIR)** – provides a general framework for modelling the relationship between rainfall and landslides. Actual rainfall data is used in this model, and it can be applied to real-time prediction for landslide occurrence when used in conjunction with a rainfall stochastic generator.
- **Saturated Unsaturated Simulation for Hillslope Instability (SUSHI)** – local model with two modules: (1) hydrological-hydraulic module for investigation of subsoil water circulation; and (2) geotechnical module for evaluation of slope stability.

Future research to establish rainfall thresholds can then be linked to alarm systems, such as electrical circuitry that is activated by a debris slide or rockfall event. Appropriate techniques are detailed in various well-known references (for example, Turner and Schuster, 1996; Wyllie and Mah, 2004), and do not form part of this thesis project. Such monitoring and alarm systems are well established in both mining and civil engineering practice, and site-specific development of alarm systems is considered appropriate for the Lower Buller Gorge rail corridor in conjunction with the establishment of threshold levels of landslide triggering.

Emergency plans and established decision-making procedures are the final component for an effective early warning system. These components are critical to ensure any system implemented is acted upon immediately in the event of landslide initiation. Procedures need to be in place to respond and mitigate the hazard. For the project area, it has been identified already that having dedicated plant available at the Westport KiwiRail depot for landslide clearance, or other emergency works, is recommended.

## 8.4 Protection at track level

From a rail management perspective, the landslide volume is just as critical for smaller events ( $<100\text{m}^3$ ) blocking the track as the comparatively larger, but less frequent events. A train travelling at a maximum speed of 50km/hour will strike the material in areas with very limited visibility regardless of the volume. The catch berm for debris on the inside of the track is regularly  $\leq 2\text{m}$  (from the nearest rail), and partial blockage of the track has therefore to be anticipated as a normal operating condition. Relevant engineering solutions that can be used at track level to control debris accumulation onto the rail are: (1) The use of catch ditches, bunds or benches to limit debris reaching the formation itself; or (2) Construction of engineered sheds or avalanche-type shelter structures.

The latter option is expensive and site-specific, but has been quite widely used in North America and Europe (Turner and Schuster, 1996). Aside from Sinclair Castle where debris preferentially travels down defined gullies, it is unlikely that such an engineering solution could be justified on a cost-benefit basis. Given the steep and narrow nature of the Lower Buller Gorge it is also unlikely that it would be feasible to construct ditches or bunds, except very locally where there was adequate width for construction. Benching into the immediate slope is likely to aggravate instability in most cases by undercutting the regolith-covered slope, and the use of corridor maintenance, based on a landslide triggering threshold approach, is favoured as the most practical risk management method.

## 8.5 Modification of slope

This technique is well documented in research dealing with rock mechanics, soil mechanics, and slope stability, and is at least technically feasible in the Lower Buller Gorge. Slope stabilisation can be broadly classified under three main methods:

- Removal of source regolith materials or loose rock by excavation and benching into slope;
- Slope drainage measures, either using site-specific horizontal drains or surface slot drains; or
- Use of stabilisation techniques, including rockbolting, buttressing or slope reinforcement.

In very localised situations these techniques can be adopted, and have recently been used effectively at Sinclair Castle where loosened rock was removed manually from upper slope source areas. The tunnel constructed above Cascade to divert a watercourse is a further example of the use of such methods, but provision is also required for long-term maintenance once implemented. Given that access for earthmoving machinery does not exist to sites above the rail at many locations, and the dependence on rail-mounted equipment, it is considered that slope modification will only be feasible at a limited number of sites if or when required.



## CHAPTER 9: SUMMARY AND CONCLUSIONS

### 9.1 Thesis objectives and methodology

Landslide hazards are recognised in the Lower Buller Gorge rail corridor that have impacted rail operations in the past, and will continue to do so in the future. The instigating factor for conducting this research was an interest from KiwiRail in determining whether two 25km/hour permanent speed restrictions applied for short lengths of rail at Whitecliffs and Te Kuha could be removed.

This thesis has incorporated these two areas into an overall landslide risk management approach and evaluation for the entire 30km length of rail between SNL96 and 126km in the Lower Buller Gorge. A framework for landslide risk management outlined by AGS (2007) and Fell et al (2008a) has been used as a guideline document to ensure consistency with international best practice.

### 9.2 Landslide risk management

The project area is characterised by high annual rainfall ( $>2,000\text{mm}$  per year), and steep topography (slopes typically  $\geq 20^\circ$ ) adjacent to the rail corridor that is susceptible to rainfall-induced landsliding. The Lower Buller Gorge is also located in a seismically active area, characterised by reverse faults. Two large earthquakes occurred in close proximity to the eastern extent of the project area in 1929 (Buller or Murchison, magnitude 7.8) and 1968 (Inangahua, magnitude 7.1).

The 30km rail corridor through the Lower Buller Gorge is located on the north bank of the Buller River, which encompasses various geological units, including basement sedimentary rocks (mapped as Greenland Group), various granites, and sedimentary breccias (Hawks Crag Breccia). Tertiary sedimentary rocks are present at the eastern end of the project area, including limestone that forms the outcrop at Whitecliffs, around SNL97.5km. Quaternary alluvial deposits, sourced from the Buller River and its catchment, form the most recent geological units.

#### 9.2.1 Landslide inventory

The main limitation identified during the research for this thesis was the absence of consistent record-keeping of landslide occurrences over time in the Lower Buller Gorge. Since the rail corridor became fully operational in the 1940s, 60 landslide events have been identified that are associated with specific location references. The information obtained enabled a qualitative assessment of landslide susceptibility to be applied to the rail corridor, but the absence of reliable temporal and frequency data restricted the ability to quantify the landslide hazard.

The volume of landslide material that can impact rail operations from upslope sources can range from a single rock to many thousands of cubic metres. To reflect the presence of comparatively smaller magnitude events, a project-specific logarithmic classification of landslides based on volume was adopted. Very small landslides are classified as being  $<10\text{m}^3$ ; small  $10\text{-}100\text{m}^3$ ; medium  $100\text{-}1,000\text{m}^3$ , large  $1,000\text{-}10,000\text{m}^3$ ; and, very large  $10,000\text{m}^3$ . Susceptibility mapping in Chapter 4 further reduced these categories to small ( $<100\text{m}^3$ ), medium ( $100\text{-}1,000\text{m}^3$ ), and large ( $>1,000\text{m}^3$ ).

### 9.2.2 Landslide susceptibility

The rail corridor can be broken into four segments based on the dominant geological unit present, topographic features, and the inventory of landslide occurrence. This has served as the basis for qualitative landslide susceptibility zonation, as follows:

- **SNL96.0 – 99.0km: Moderate landslide susceptibility** - dominant flat-lying indurated limestones, with sub-vertical joint-controlled cliff escarpments to 80m in height and the development of a steep ( $35\text{-}50^\circ$ ) rock talus apron).
- **SNL99.0 – 103.5km: Low landslide susceptibility** – alluvial floodplains of the Mackley River and Muddy Creek.
- **SNL103.5 – 110.0km: Moderate to high landslide susceptibility** – steep topography (typically  $\geq 20^\circ$ , with some areas  $\geq 35^\circ$ ) developed predominantly on Greenland Group bedrock and bedded Kaiata Formation mudstones and sandstones.
- **SNL110.0 – 126.0km: High landslide susceptibility** – steep topography (typically  $\geq 35^\circ$ ) developed predominantly on granitic rocks, breccias and low rank meta-sediments, with minor schist.

The assessment based on landslide susceptibility has shown that all large ( $>1,000\text{m}^3$ ) landslides have developed from the steep ( $\geq 20^\circ$ ) slopes west of SNL107.5km, and that there is no direct correlation with any of the lithologies present. Only one medium volume ( $100\text{-}1,000\text{m}^3$ ) landslide has been identified east of this location (near SNL96km), while small and very small ( $<100\text{m}^3$ ) landslides are distributed essentially throughout the 30km rail corridor. The floodplains of the Mackley River and Muddy Creek, between SNL99 and 103.5km, as expected show no significant landsliding.

### 9.2.3 Landslide hazard

Rainfall-induced and earthquake-generated slope failures both have the potential to impact rail infrastructure within the project area. Approximately 78% by length of the rail corridor (SNL96.0-97.0km and 103.5-126.0km) is subject to shallow translational landslides developed in typically 2-3m thick regolith-colluvium overlying strong, jointed, and relatively unweathered bedrock. Volumes

range from  $<10\text{m}^3$  (very small landslides) to some exceeding  $10,000\text{m}^3$  (very large landslides), and are triggered either by rainfall events or seismicity.

Bedrock failures are limited to joint-controlled block movements generally less than  $10\text{m}^3$  in volume. At Sinclair Castle (SNL118.8km) and Cascade (SNL121km), bedrock blocks from the upslope source areas periodically mobilise as channelised debris that have historically impacted rail operations, and can be expected to continue in response to climatic triggers.

### 9.3 Speed restriction zone – Whitecliffs

Concerns about open jointing in the rock mass at Whitecliffs (SNL97 to 99km) have resulted in a short speed restriction being applied, which has been evaluated as part of this thesis. There does not appear to be any documentation or institutional knowledge as to the timing or specific purpose of the nominal 20m restriction length. A comprehensive literature review related to impacts on the rail corridor from the 1968 Inangahua Earthquake, in conjunction with detailed historic aerial photograph reviews, has established that no failures originated from the pre-existing open joint-controlled fractures in the limestone cliff. It is recognised that some dilation of these joint-controlled fractures likely occurred. Earthquake-triggering of fracture-controlled block failures up to  $\sim 1,000\text{m}^3$  in volume are considered geotechnically feasible based on measurements derived from the southernmost fracture identified closest to the rail corridor (Fracture 3).

Detailed mapping and logging, including two abseil traverses down the 65m high limestone cliffs, as well as sampling and materials testing, has established that there is no evidence for active slope movement. The assessment undertaken has shown that a combination of bedding dip into the face, relatively high intact strength ( $>40\text{MPa}$ ), widely-spaced ( $\geq 10\text{m}$ ) sub-vertical joints, and stable pin arrays since implementation in 1993, together provide confidence in long-term stability. It is recommended that the speed restriction zone be removed, and consideration given to a real-time early warning system based on monitoring of a network of extensometers or similar.

### 9.4 Speed restriction zone – Te Kuha

A 450m long 25km/hour speed restriction zone has been established at Te Kuha between SNL124.05 and 124.50km. There is no knowledge or documentation as to the reasons for this zone, but it is likely related to the large ( $\sim 18,000\text{m}^3$ ) landslide that occurred in 1987 at Windy Point (SNL125.4km). The 2km section at Te Kuha between SNL124.0 and 126.0km was evaluated by field mapping, Schmidt Hammer testing, sampling and laboratory evaluation of the exposed granite (with minor schistose Greenland Group inclusions).

Slopes are steep ( $\geq 35^\circ$ ) for much of this part of the rail corridor, and the strong and relatively unweathered bedrock is covered by regolith-colluvium typically 2-3m thick. Three very large but shallow translational landslides  $\geq 10,000\text{m}^3$  in volume have occurred since 1987, affecting the rail operations and exposing stable bedrock. Smaller landslides, including some joint-controlled failures in bedrock  $\leq 10\text{m}^3$  in volume, have also impacted the rail corridor at Te Kuha, and there is potential for on-going rainfall-induced failures at this location.

The risk analysis example undertaken indicates a relatively high annual probability of slope failures at Te Kuha (of the order of  $10^{-3}$ ), but these can be expected over a distance of some 1,120m (or 56% of the corridor length evaluated). It is concluded that continuation of the speed restriction is not justified, as much of the shallow debris within the 450m section has been removed by landsliding in the past 20 years. It is recommended that the speed restriction be removed, and that proactive management be further implemented based around adoption of a rainfall threshold approach to minimise future risk.

## 9.5 Future landslide risk management

The principal conclusion from this thesis project is that there is on-going risk to rail operations from shallow translational landsliding in regolith-colluvium materials at numerous sites from SNL96.0-97.0km, and SNL107.5-126.0km. The majority of these will be generated by long-duration or intense rainfall events. Development of a threshold-based method for effective track management is recommended, including the establishment of a rain gauge network through the Lower Buller Gorge. Although track protection, maintenance and related measures remain important for rail management, high capital cost measures such as benching, bunding or avalanche-type shelters are unlikely to be justified on a cost-benefit basis.

At Whitecliffs the implementation of a real-time warning system based around measurement of fracture dilation or movement is considered appropriate. Further modelling and evaluation of seismic response to large magnitude earthquake events may assist, but as with the rainfall-induced landslide triggering threshold approach elsewhere in the gorge, protocols for temporary line closure and immediate inspection are considered to be the most appropriate. It is therefore recommended that proactive identification of potential hazard areas based on an inventory system be adopted, and that this be compiled into a database and available for both long-term analysis and day-to-day management requirements.

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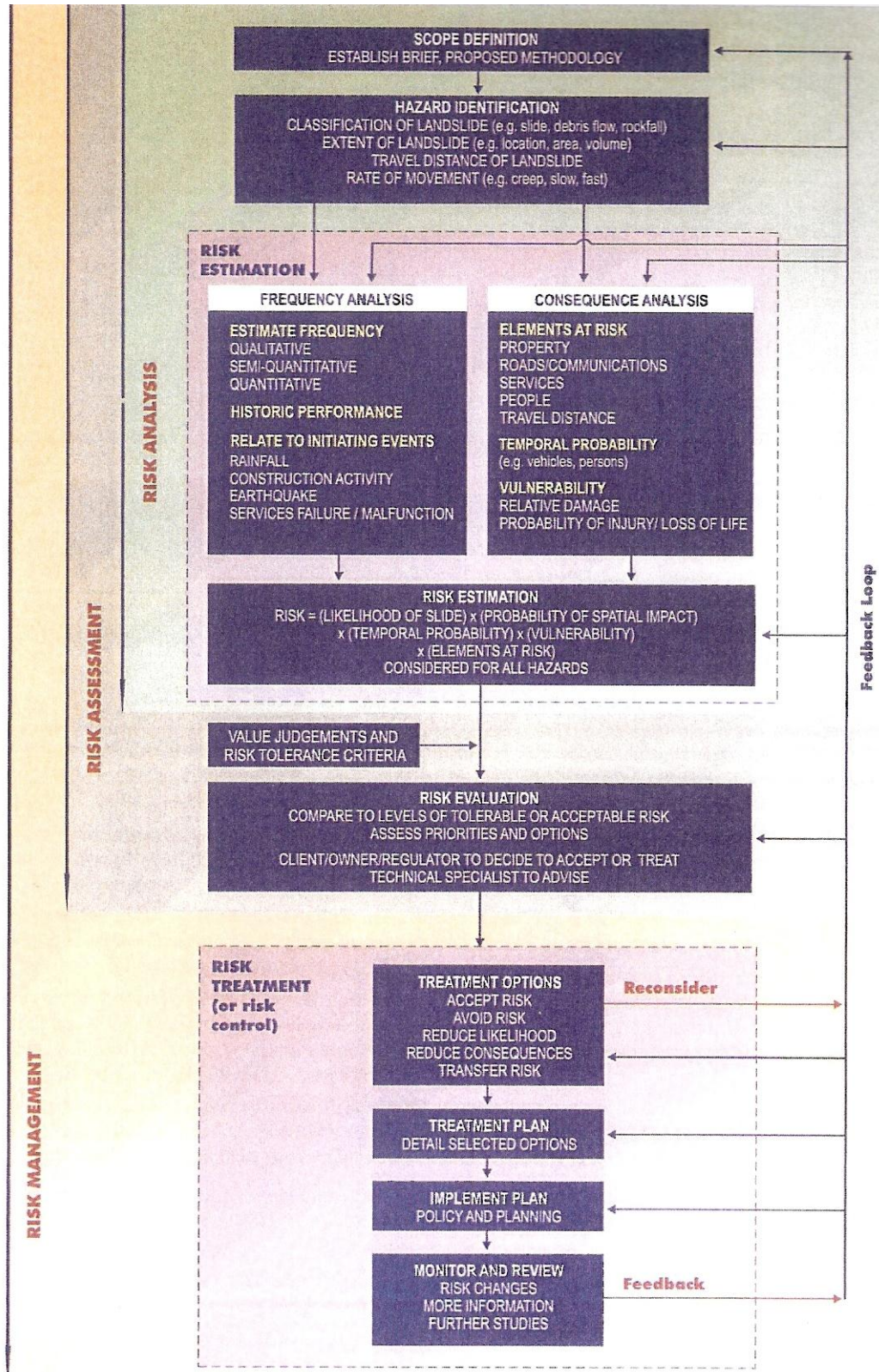
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# Appendix 1.1

*Flowchart for Landslide Risk Management (AGS, 2007, pg 6)*



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## Appendix 1.2

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### *Definitions of terms used by Fell et al (2008a) in regards to landslide zoning and risk management*

**Acceptable risk** — A risk for which, for the purposes of life or work, society is prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

**Annual Exceedance Probability (AEP)** — The estimated probability that an event of specified magnitude will be exceeded in any year.

**Consequence** — The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

**Danger** — The natural phenomenon that could lead to damage, described in terms of its geometry, mechanical and other characteristics. The danger can be an existing one (such as a creeping slope) or a potential one (such as a rock fall). The characterisation of a danger does not include any forecasting.

**Elements at risk** — The population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

**Frequency** — A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

**Hazard** — A condition with the potential for causing an undesirable consequence. The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the probability of their occurrence within a given period of time.

**Individual risk to life** — The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

**Landslide inventory** — An inventory of the location, classification, volume, activity and date of occurrence of landsliding.

**Landslide activity** — The stage of development of a landslide; prefailure when the slope is strained throughout but is essentially intact; failure characterized by the formation of a continuous surface of rupture; post-failure which includes movement from just after failure to when it essentially stops; and reactivation when the slope slides along one or several pre-existing surfaces of rupture. Reactivation may be occasional (e.g. seasonal) or continuous (in which case the slide is “active”).

**Landslide intensity** — A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

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**Landslide susceptibility** — A quantitative or qualitative assessment of the classification, volume (or area) and spatial distribution of landslides which exist or potentially may occur in an area. Susceptibility may also include a description of the velocity and intensity of the existing or potential landsliding.

**Likelihood** — Used as a qualitative description of probability or frequency.

**Probability** — A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. There are two main interpretations:

- i. Statistical-frequency or fraction — The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an “objective” or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.
- ii. Subjective probability (degree of belief) — Quantified measure of belief, judgement, or confidence in the likelihood of a outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgement regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.

**Qualitative risk analysis** — An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.

**Quantitative risk analysis** — An analysis based on numerical values of the probability, vulnerability and consequences, and resulting in a numerical value of the risk.

**Risk** — A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability×consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

**Risk analysis** — The use of available information to estimate the risk to individuals, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: Scope definition, hazard identification, and risk estimation.

**Risk assessment** — The process of risk analysis and risk evaluation.

**Risk control or risk treatment** — The process of decision making for managing risk, and the implementation or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

**Risk estimation** — The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

**Risk evaluation** — The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

**Risk management** — The complete process of risk assessment and risk control (or risk treatment).

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**Societal risk** — The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

**Susceptibility** — see Landslide susceptibility.

**Temporal-spatial probability** — The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

**Tolerable risk** — A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.

**Vulnerability** — The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

**Zoning** — The division of land into homogeneous areas or domains and their ranking according to degrees of actual or potential landslide susceptibility, hazard or risk.



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## Appendix 1.3

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### *KiwiRail monitoring programme and results at Whitecliffs between 1993 and 2011*

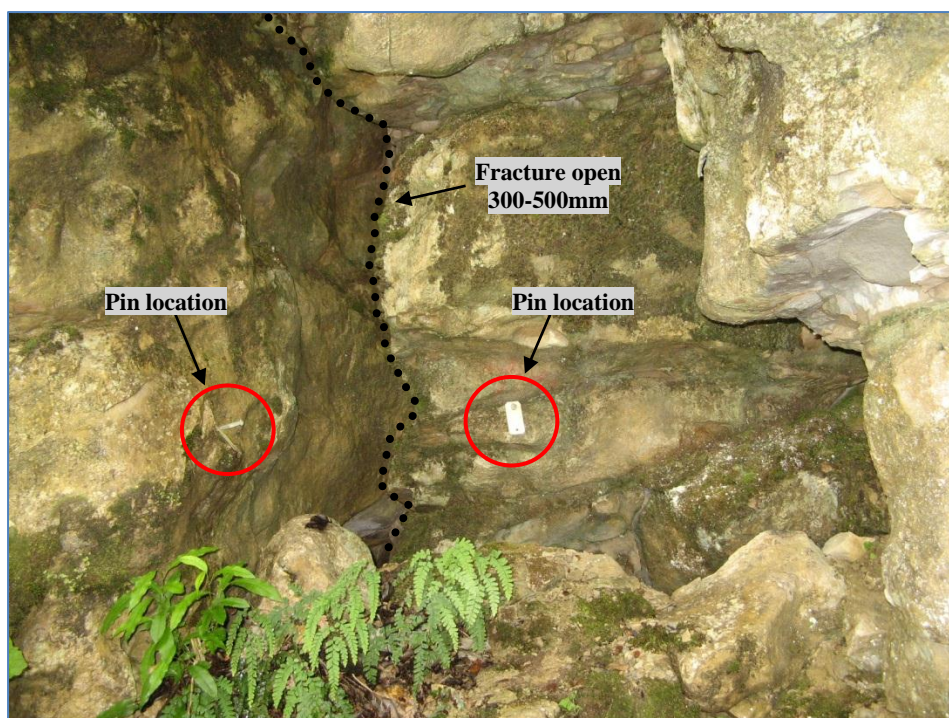
Metal pins are located across selected displaced limestone blocks at the top of Whitecliffs for the purpose of monitoring any gradual movement over time. Figures 1.3A and 1.3B show the typical environment at the top of Whitecliffs where the pins are located, marked with metal tags. Displacement monitoring has generally been undertaken annually since December 2001, with the first monitoring round conducted in January 1993. A tape measure is used to measure the distance between the centres of each set of pins. A summary of the measurements taken to date is provided in Table 1.3A, which indicates there was no gradual block displacement between January 1993 and February 2011. The method adopted for the monitoring has an error of up to approximately  $\pm 14\text{mm}$ , with results more typically in a range of  $\pm 1$  to  $6\text{mm}$  between monitoring rounds.

The frequency of monitoring and absence of telemetered information will not enable an early warning of large scale cliff collapse but has been useful to show that the seemingly precarious blocks at the top of Whitecliffs are relatively stable under current environmental and climatic conditions for at least the past 18 years.



**Figure 1.3A: Top of Whitecliffs (elevation ~140m above mean sea level). Limestone block displacement monitoring location**





**Figure 1.3B: Limestone block displacement monitoring location**

**Table 1.3A: Pin measurements recorded between 1993 and 2011**

Monitoring date	Pin set references and measurement (mm)					
	A1 - A2	B1 - B2	B2 - D1	C1 - D1	C1 - C2	C2 - D1
January 1993	990	1200	2165	1765	792	1665
December 2001	996	1202	2164	1765	791	1662
December 2002	998	1202	2167	1767	792	1662
December 2004	990	1200	2170	1764	791	1662
February 2007	988	1200	2166	1764	792	1661
December 2008	984	1200	2163	1764	791	1662
August 2009	984	1200	2161	1764	790	1662
September 2010	984	1202	2156	1762	792	1655
February 2011	985	1200	2164	1768	792	1663
Minimum measurement to Feb-2011	984	1200	2156	1762	790	1655
Maximum measurement to Feb-2011	998	1202	2170	1768	792	1665
Maximum difference between all rounds (mm)	-14	-2	-14	-6	-2	-10
Difference between Jan-1993 and Feb-2011 (mm)	5	0	1	-3	0	2

# Appendix 1.4

## *Glossary for forming names of landslides from Cruden and Varnes (1996)*

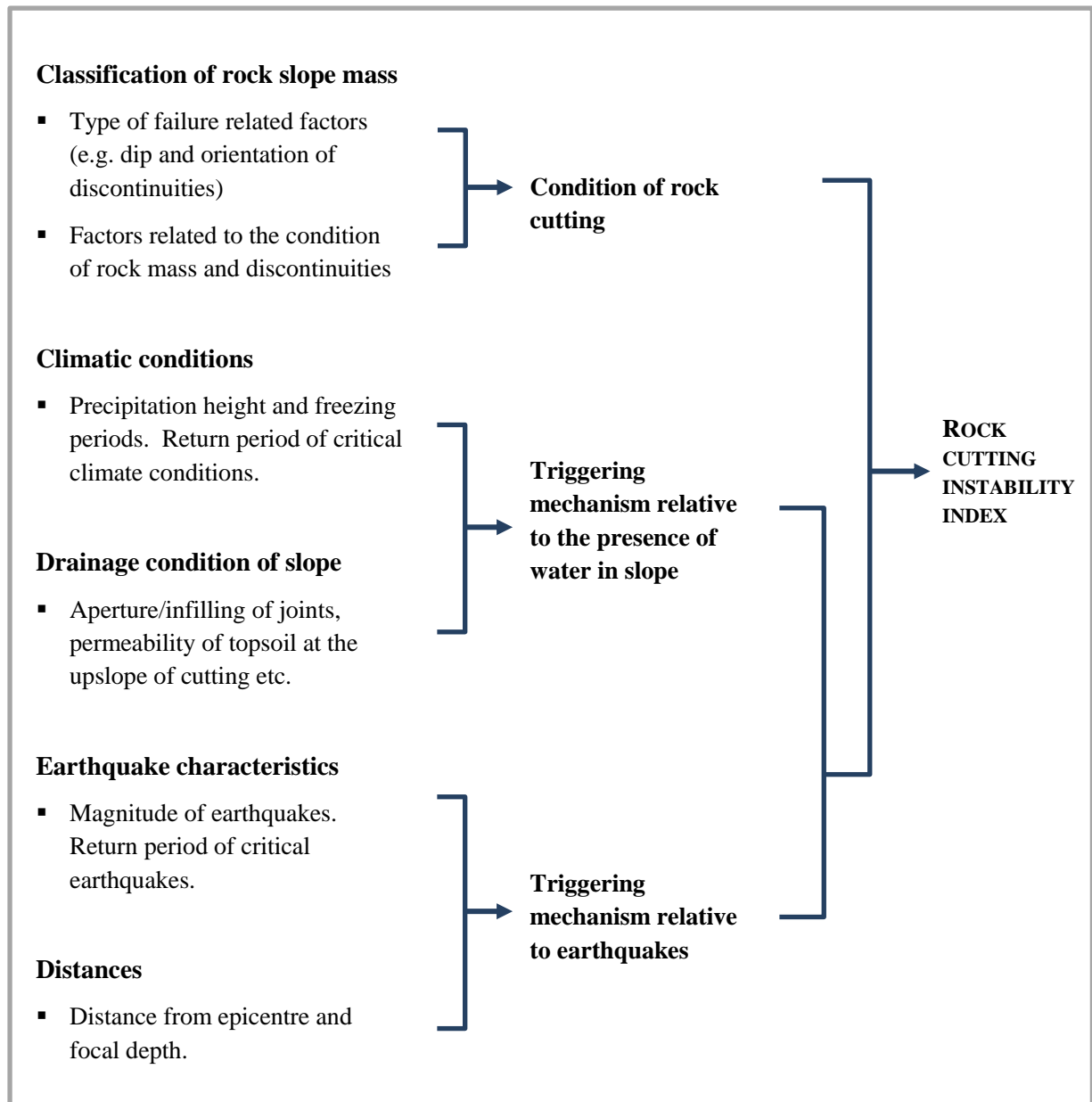
**Table 1.4A: Glossary for Forming Names of Landslides (reproduced from Cruden and Varnes, 1996, p38)**

ACTIVITY			
STATE	DISTRIBUTION	STYLE	
Active	Advancing	Complex	
Reactivated	Retrogressive	Composite	
Suspended	Widening	Multiple	
Inactive	Enlarging	Successive	
- dormant	Confined	Single	
- abandoned	Diminishing		
- stabilised	Moving		
- relict			
DESCRIPTION OF FIRST MOVEMENT			
RATE	WATER CONTENT	MATERIAL	TYPE
Extremely rapid	Dry	Rock	Fall
Very rapid	Moist	Soil	Topple
Moderate	Wet	- earth	Slide
Slow	Very wet	- debris	Spread
Very slow			Flow
Extremely slow			
DESCRIPTION OF SECOND MOVEMENT			
RATE	WATER CONTENT	MATERIAL	TYPE
Extremely rapid	Dry	Rock	Fall
Very rapid	Moist	Soil	Topple
Moderate	Wet	- earth	Slide
Slow	Very wet	- debris	Spread
Very slow			Flow
Extremely slow			

Note: subsequent movements may be described by repeating the above descriptors as many times as necessary

# Appendix 1.5

## *Rock mass classification system from Pantelidis (2009)*



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## Appendices 3.1 to 3.4

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*Literature references relevant to this appendix are incorporated within the main references section  
(Pages 151 to 156)*

A3.1	West Coast Regional Council records .....	Page 1
A3.2	Historic aerial photography review .....	Page 5
A3.3	1929 Buller earthquake.....	Page 12
A3.4	1968 Inangahua earthquake .....	Page 14

**Note:** The following sections are based on information sourced to complement the limited landslide inventory data available from KiwiRail. The structure of Appendices 3.1 to 3.4 follows that defined in Chapter 3, Table 3.1.

### A3.1 West Coast Regional Council records

DTec Consulting Limited (2002) prepared a report for the West Coast Regional Council (WCRC) that provides a general overview of natural hazards relevant to the West Coast Region of New Zealand. This report was reviewed as it includes two inventories (flooding and landslide events) specific to the region. Both inventories are noted as incomplete and further research was recommended at the time of the publication. The majority of entries are based on a detailed search of the Greymouth Evening Star news publication and Benn (1990).

Each entry in the two inventories was reviewed to determine the frequency of reported incidents specific to the rail corridor in the Lower Buller Gorge. Recorded landslides and flooding events often only refer to the road infrastructure. The occurrence of landslides impacting the Lower Buller Gorge is unlikely to always be confined to one side of the Buller River only. Accordingly, the summary table of reported incidents (Table A3.1.1) includes detail specific to the rail corridor and SH6 between Inangahua Junction and Westport. The information is based directly on entries from DTec Consulting Limited (2002) and is presented in chronological order between 1925 and 2002, including the incident date, triggering event/cause and detail available. The rail corridor was only fully completed in the 1940s so entries up to this date are included for determining the frequency of events, despite there being potentially no impacts to rail operations.

**Table A3.1.1: Flooding and landslide inventory for the Lower Buller Gorge (road and rail transportation corridors) between 1925 and 2003 (based on DTec Consulting Limited, 2002)**

Date	Detail
22 September 1925	Cause: <b>Rain</b> (two weeks of rain in Buller). Roads around Westport and Buller blocked by slips and washouts. Westport completely isolated.
30-31 October 1926	Cause: <b>Rain</b> . Slips blocked roads in Westland and considerable damage to roads in the Buller Gorge (SH6).
16 June 1929	Cause: <b>Buller earthquake</b> (M7.8, MM10-11 in epicentral region). Severe slips throughout region, especially in the upper Buller catchment. Many rivers blocked by slips forming earthquake dammed lakes.
7-10 October 1930	Cause: <b>Rain</b> . Many slips in the Westport area.
3 April 1931	Cause: <b>Rain</b> . Many roads blocked by slips. Fatality at Tiroroa (SH6).



Date	Detail
20 February 1935	Cause: <a href="#">Rain</a> . Slips (and washouts) caused severe road damage throughout the West Coast.
8-9 January 1938	Cause: <a href="#">Rain</a> . Slips (and washouts) blocked many main roads in Buller.
4-8 April 1942	Cause: <a href="#">Rain</a> . Numerous slips on roads throughout the region, including Buller, Westport to Murchison.
12-13 July 1942	Cause: <a href="#">Rain</a> . Several coalmines idle due to slips on their rail lines. Buller Gorge Road (SH6) blocked by slips.
10 February 1944	Cause: <a href="#">Rain</a> . Slips on rail line in Buller Gorge. Train ran into slip 20km from Westport, losing 13 wagons of coal.
29 July 1944	Cause: <a href="#">Rain</a> . Buller Gorge Road (SH6) blocked by several slips.
16-19 February 1955	Cause: <a href="#">Rain</a> (Inangahua - 90mm in <48 hours). Slips blocked road between Inangahua Junction and Westport. Buller Gorge Road (SH6) blocked by slips at Husband Hill. Rail car ran into a slip in the Buller Gorge on the 18th, no injuries.
26-27 February 1955	Cause: <a href="#">Rain</a> . Widespread serious slip damage. Buller Gorge Road (SH6) blocked by slips. All rail services to coast cut by slips (and flooding). Slips caused serious damage to the Cascade Mine at Denniston - buildings and equipment was buried and over 11km of fluming leading to the mine was carried away by slips.
7-10 November 1961	Cause: <a href="#">Rain</a> . SH6 in the Buller Gorge cut by slips and floodwater. Buller Gorge rail line cut by slips.
24-25 March 1964	Cause: <a href="#">Rain</a> . Slips blocked Buller Gorge railway 28km west of Inangahua.
26-27 April 1966	Cause: <a href="#">Rain</a> (Westport - 63mm and Inangahua - 51mm in 24hrs). Slip closed the Buller Gorge railway near Rahui.
10-12 March 1967	Cause: <a href="#">Rain</a> . Upper and Lower Buller Gorge roads (SH6) closed due to slips.
26-27 April 1967	Cause: <a href="#">Rain</a> . Slips closed SH6 between Inangahua and Westport, and between Tiroroa and Rahui.
2-4 July 1967	Cause: <a href="#">Rain</a> (Inangahua - 73mm). A few minor slips in the Buller Gorge.
24 May 1968	Cause: <a href="#">Inangahua earthquake</a> (M7.1, MM10). Thousands of slips throughout the region. Buller District and Buller Gorge severely affected. All roads and rail links around Inangahua and through the Buller Gorge blocked by slips. Slip killed a person at Whitecliffs in the Buller Gorge.

Date	Detail
11-13 August 1968	Cause: <b>Rain</b> . Upper and Lower Buller Gorge roads closed due to slips – likely re-activation of Inangahua earthquake slips.
22-23 October 1968	Cause: <b>Rain</b> . Rain re-activated Inangahua earthquake slip at the Cascade Mine and blocked Cascade Creek. Creek backed up and flooded mine causing considerable damage to plant (\$2,000). Bins, compressor electric pumps and other equipment lost.
28-30 October 1968	Cause: <b>Rain</b> . Many roads around Westport cut by slips (and floodwaters), as were rail links.
28-31 August 1970	Cause: <b>Rain</b> . Slips blocked rail links between Greymouth and Westport and closed roads in the Buller Gorge (SH6).
17 September 1970	Cause: <b>Rain</b> . Most major highways affected by slips and flooding. Lower Buller Gorge (SH6) closed by slips and floods.
29-31 March 1975	Cause: <b>Rain</b> (Westport - 80mm/Buller Gorge - 130mm in 24 hours). Lower Buller Gorge (SH6) covered by many slips. Westport to Inangahua section closed for 36 hours.
18-19 January 1977	Cause: <b>Rain</b> . Slips blocked the rail line between Tiroroa and Westport.
13-14 April 1978	Cause: <b>Rain</b> . Numerous slips cut rail links.
3 December 1979	Cause: <b>Rain</b> . Slips blocked the Buller Gorge rail line.
29 April 1981	Cause: <b>Rain</b> . Many slips reported in Inangahua Junction and Ngakawau.
14-20 January 1987	Cause: <b>Rain</b> . Slips caused major disruption to transport routes. A major slip destroyed half of the Windy Point rail tunnel in the Buller Gorge on the 20th.
28-29 January 1987	Cause: <b>Rain</b> . Windy Point rail tunnel again blocked by slips.
3-4 February 1987	Cause: <b>Rain</b> . Buller Gorge rail again closed by slips at Windy Point Tunnel.
19-20 May 1988	Cause: <b>Rain</b> (prolonged heavy rain after driest April on record). Numerous slips throughout region - three slips cut the Westport to Greymouth rail line.
29 January 1991	Cause: <b>Westport earthquakes</b> (M6.0 and M6.1 - two distinct shocks). Major slip (80m long) blocked SH6 at Tiroroa, along with numerous slips in the Buller Gorge.
15 February 1991	Cause: <b>Westport earthquake</b> (M5.8). Minor rockfalls in the Buller Gorge.
7 August 1991	Cause: <b>Rain</b> . Slips closed SH6 for short periods, including the Lower Buller Gorge.

Date	Detail
13-14 June 1993	Cause: <a href="#">Rain</a> . Major flooding in Buller District. Buller River levels highest since 1970s. Buller discharged $7,800\text{m}^3\text{s}^{-1}$ , peaking at 5:23pm and coinciding with high tide. Westport sandbagged, Buller Gorge road (SH6) submerged under 2m of water in places, and the lower Buller Gorge closed on the 13th and 14th. River flowing very fast and full of logs.
7-8 November 1994	Cause: <a href="#">Rain</a> . Widespread flooding. The Buller River peaked at 10.7m at Te Kuha, 8.7m above normal. Flooding closed SH6 at Inangahua Junction (water 1.5m deep on road).
12-23 November 1994	Cause: <a href="#">Rain</a> . Buller hardest hit. First Civil Defence emergency in the Buller District since the Inangahua earthquake in 1968. Buller river peaked at 10.7m at Te Kuha. SH6 closed by floodwaters at Hawk's Crag in the Buller Gorge – 0.5m of water over the road, and between Inangahua Junction and Inangahua Camp.
26 September 1995	Cause: <a href="#">Rain</a> . Buller River 6m above normal.
15-16 December 1997	Cause: <a href="#">Rain</a> . SH6 closed at Inangahua Junction and Hawk's Crag due to flooding.
1-3 July 1998	Cause: <a href="#">Rain</a> . SH6 closed for 12 hours at Inangahua Junction due to flooding.
20 October 1998	Cause: <a href="#">Rain</a> . SH6 closed at Inangahua Junction.
27-29 October 1998	Cause: <a href="#">Rain</a> . The Buller River reached road level at Hawk's Crag. 900mm of water flooded SH6 at Inangahua Junction.
19-20 March 2002	Cause: <a href="#">Rain</a> . SH6 in the Buller Gorge closed due to flooding.

The inventory presents general event details only. There is no information regarding exact locations of landslides, type of slope failure or volumes of material released. Despite the lack of technical detail, the inventory does highlight the challenges faced in the region due to high annual rainfall and being located in a seismically active area. Of the 46 entries over the 78 year period between 1925 and 2003, 42 of these list rain as the triggering factor with the remaining four entries relating to seismicity. There are no temporal trends evident from the data. Only one entry refers specifically to rockfalls in the Buller Gorge (as a result of a magnitude 5.8 Westport earthquake on 29 January 1991).

Research into triggering levels for initiating slope movement in response to rainfall has not been quantified to date in the Lower Buller Gorge, or within the wider region. This was an area that DTec Consulting Limited (2002) identified as requiring further research.

## A3.2 Historic aerial photography review

To understand the geomorphic changes in landform appearance over time, aerial photographs are a useful tool for determining any large-scale slope movement features. In terms of developing a landslide inventory, aerial photographs are a key component to ensure a robust assessment of historic data is presented. Historic aerial photographs were ordered from NZAM for this purpose. Stereopair contact prints obtained are detailed in Table A3.2.1. The photograph sets were chosen based on complete project area coverage, scale and range of dates flown.

**Table A3.2.1: NZAM aerial photographs**

Date flown	Scale	Survey reference	Photograph numbers	Detail
1 November 1947	1:16,000	SN265	1459/5 to 1459/10 1460/11 to 1460/21 1459/22 to 1459/27	23 contact prints, covering entire project area.
7 October 1959	1:44,000	SN1249	2647/6 to 2647/15	8 contact prints, covering entire project area.
27 November 1985	1:5,500	SN8575	Q/1 to 14; R/2 to 11; S/3 to 7; T/1 to 4; U/3 to 6; V/3 to 7; W/3 to 7; X/3 to 6; Y/2 to 9; Z/3 to 4	74 contact prints, covering entire project area.

The 1947 aerial photographs represent the rail corridor as it existed not long after completion, particularly at the eastern end of the project area that was only completed in the early 1940s. The aerial photograph coverage from 1947 is shown on Figures A3.2.1 and A3.2.2 with SNL metrage locations overlain. Due to the comparatively more recent track formation between Cascade and Buller, the aerial photographs regularly exhibit land clearance adjacent to the rail corridor. The scale of 1:16,000 is adequate for viewing comparatively larger scale slope failure surfaces.

The 1959 aerial photographs also cover the entire project area and were chosen as the most recent data set available prior the 1968 Inangahua earthquake. Detail is difficult to define within the rail corridor as a result of the scale (1:44,000) and shadow effects on the rail side of the Buller River. The photographs do show progressive re-vegetation of the land adjacent to the track formation that was disturbed during construction stages and absence of any large-scale slope failures between the period 1947 and 1959.



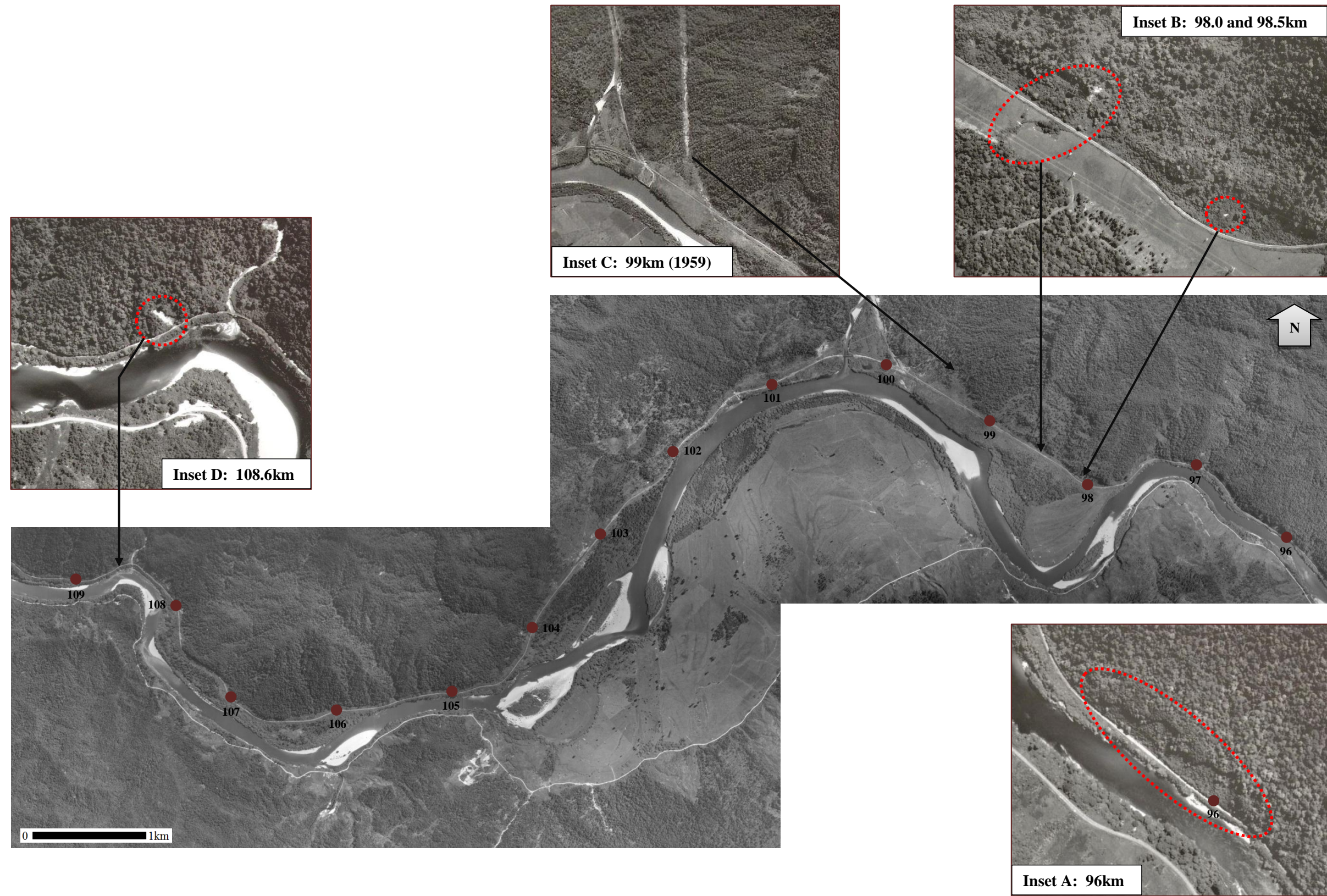


Figure A3.2.1: 1947 aerial photography between SNL96km and 109km. Refer Table A3.2.2 for inset descriptions.



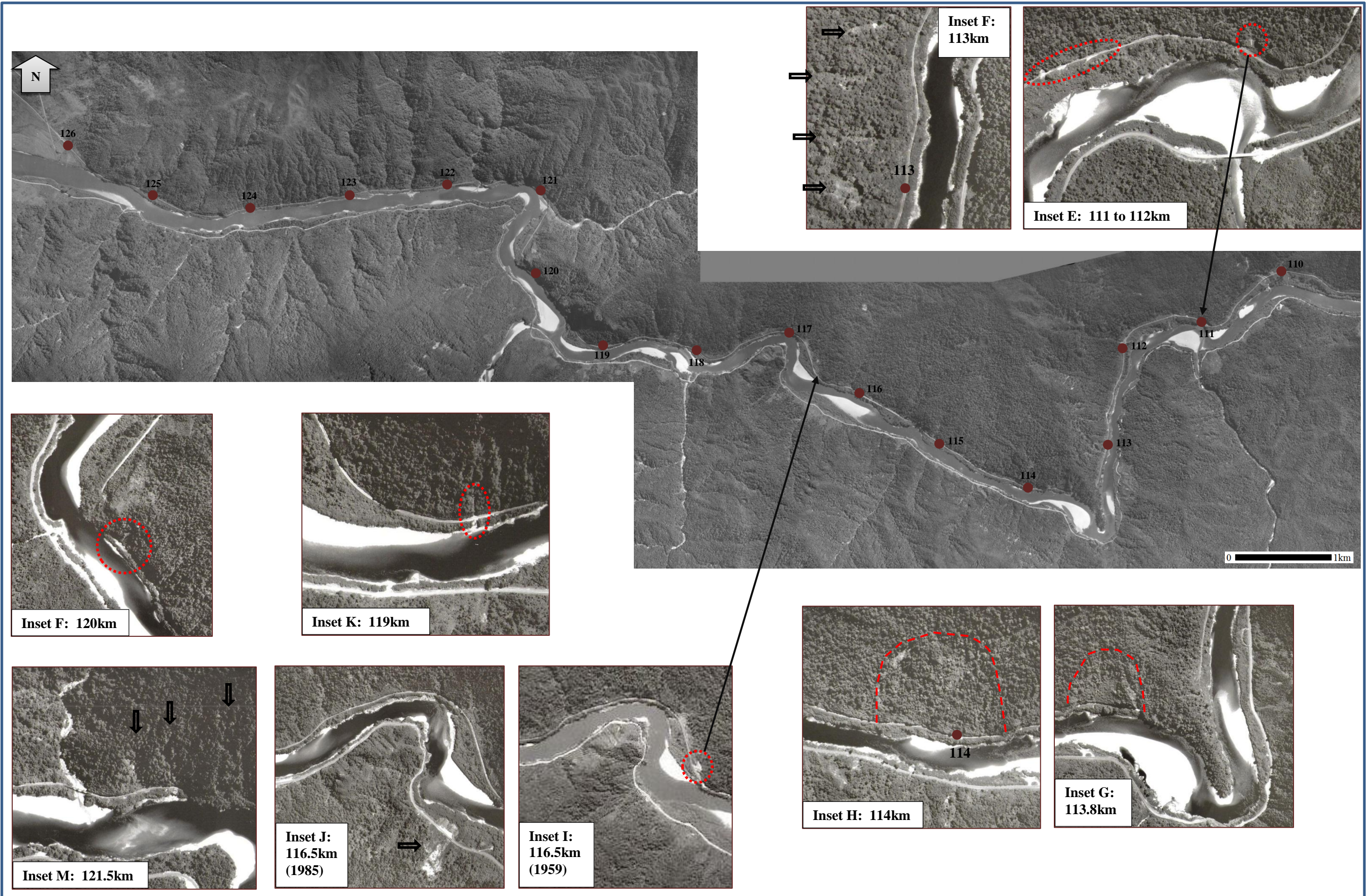


Figure A3.2.2: 1947 aerial photography between SNL110km and 126km. Refer Table A3.2.2 for inset descriptions.



The scale of the 1985 aerial photographs (1:5,500) allows a comparatively more detailed inspection of the slopes adjacent to the rail. Some areas are impacted by shadow effects but the data set from 1985 is very useful for determining landslide features that do not appear in the 1947 images. Insets on Figures A3.2.1 and A3.2.2 show examples of changes related to slope movement between the 1947 and 1985 imagery.

### A3.2.1 Aerial photograph interpretation

Detail viewed using aerial photography specific to landslides typically relates to changes in morphology, vegetation, and drainage characteristics of a particular slope (Soeters and van Westen, 1996). Stereoscopic views provide the most robust method for assessing aerial photographs and a stereoscope was used throughout the review. A summary table of terrain features, and the associated aerial photograph characteristics in terms of slope stability, is provided at the end of this appendix for reference (from Soeters and van Westen, 1996).

Different scales, shadow effects and vertical distortion in aerial photography can cause limitations in interpretation, particularly when viewing steep slopes. Despite these limitations the review identified numerous features, as detailed in Table A3.2.2. Examples of slope movement features are shown on Figure A3.2.1 (SNL96km to 109km) and Figure A3.2.2 (SNL110km to 126km). SNL metrage references are considered accurate to approximately +/-50m.

**Table A3.2.2: Aerial photograph interpretation**

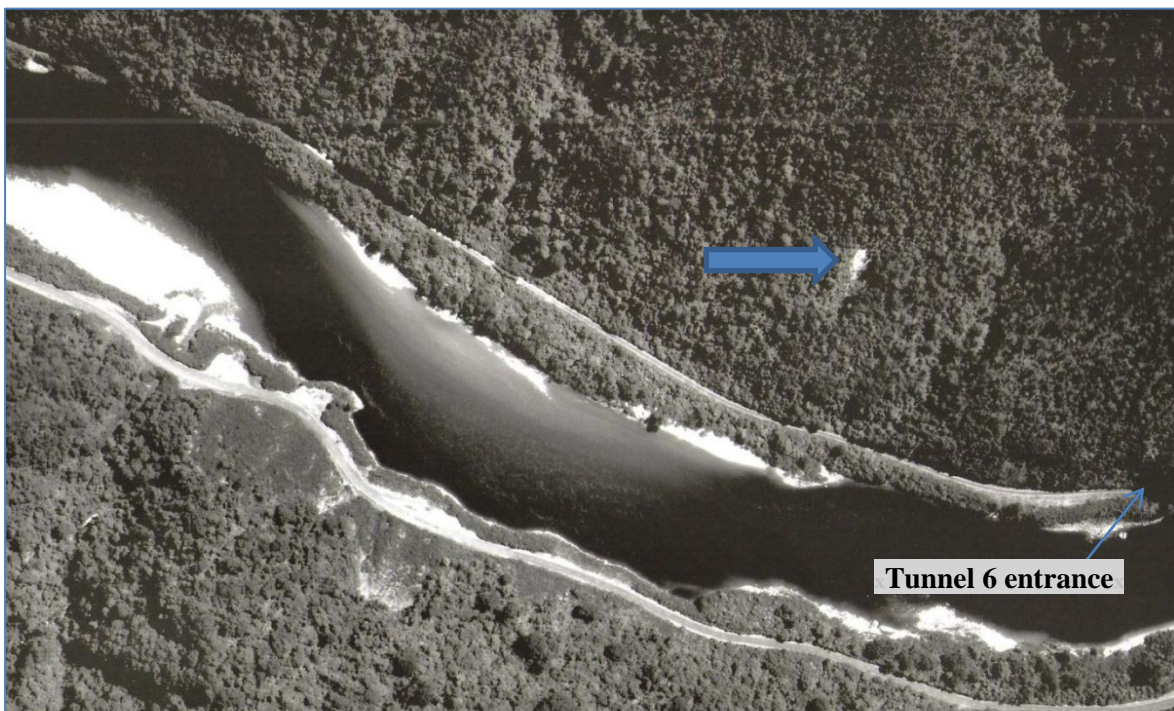
SNL metrage (km)	Description	Inset reference, date and figure
96.0	Shallow regolith failures in the vicinity of 96km have occurred between 1959 and 1985. Recent shallow, small-scale, regolith failures were also noted during field work in February 2011 (refer Chapter 4).	Inset A, 1985 (Figure A3.2.1)
97.5	No large-scale failure at Whitecliffs evident between 1947 and present day. A detailed discussion of Whitecliffs and relevant photographs is presented in Chapter 6.	No inset (Figure A3.2.1)
98.0	Unvegetated detached block evident in 1985 photograph (circled feature on right-hand side of Inset B), not visible in 1947 or 1959.	Inset B, 1985 (Figure A3.2.1)
98.5	Recent small landslide upslope of rail evident in 1985, and vegetation on southern side of rail corridor that denotes the stream location has become more dense (left-hand side of Inset B).	Inset B, 1985 (Figure A3.2.1)
99.0	Access road adjacent to power pylons, and the Mackley River, north of the rail corridor first evident in 1959 photographs.	Inset C, 1959 (Figure A3.2.1)

SNL metrage (km)	Description	Inset reference, date and figure
108.6	Small landslide evident to the west of Payne Creek in 1985, not visible in 1947 or 1959. Historic landslide also evident on State Highway 6 in 1985 photograph.	Inset D, 1985 (Figure A3.2.1)
111.0	Small-scale landslide above the rail at 111km in 1985. Additional small-scale features indicating slope instability are also evident moving west from 111km to 112km.	Inset E, 1985 (Figure A3.2.2)
113.0	Old (pre-1947) landslide feature evident, rail corridor has been constructed through this formation.	Inset F, 1985 (Figure A3.2.2)
113.8	Large pre-1947 landslide on western side of tunnel through Hawks Crag Breccia.	Inset G, 1985 (Figure A3.2.2)
114.0	Large pre-1947 landslide. Potentially an area that becomes reactivated as the Buller River bank profile appears consistent since 1947 but a fresh scar is evident upslope of the rail in 1985.	Inset H, 1985 (Figure A3.2.2)
116.5	Scar visible above the rail corridor in 1959 aerial photography. Also signs of slope instability on the southern side of the Buller River, adjacent to State Highway 6 in 1985, based on fresh scar in the landscape and encroachment of a landslide into the Buller River.	Inset I, 1959 Inset J, 1985 (Figure A3.2.2)
118.6 to 119.5	Slope movement features visible in 1947, 1959 and 1985 aerial photographs, associated with large historic landslide and on-going issues associated with Sinclair Castle. Inset K (Figure A3.2.2), provides an example from 1985 around 119.0km. Landslide features are also adjacent to SH6.	Inset K, 1985 (Figure A3.2.2)
120.0	Recent material disturbance adjacent to rail, possibly related to a landslide above the rail alignment.	Inset L, 1985 (Figure A3.2.2)
121.5	Slope movement near Cascade Creek on the western side of Tunnel 5 evident in 1985. No fresh scars visible in 1947 or 1959 photography.	Inset M, 1985 (Figure A3.2.2)

From around 122km to the western extent of the project area at 126km there are numerous steep gullies and streams that intersect the rail corridor. At 122.4km in 1985 the discharge from one of these streams is evident (refer arrow on Figure A3.2.3). Recent landslides are evident in 1985 that have impacted SH6 (opposite SNL125.5km, Figure A3.2.4), and a fresh scar west of the Tunnel 6 entrance.



**Figure A3.2.3: Steep gullies adjacent to the rail corridor and stream discharges creating small alluvial fans in the Buller River. Example shown from 1985 (indicated by arrow).**



**Figure A3.2.4: Small slope movement feature above ~124.50km (indicated by arrow) in 1985 and slope instability issues adjacent to SH6.**



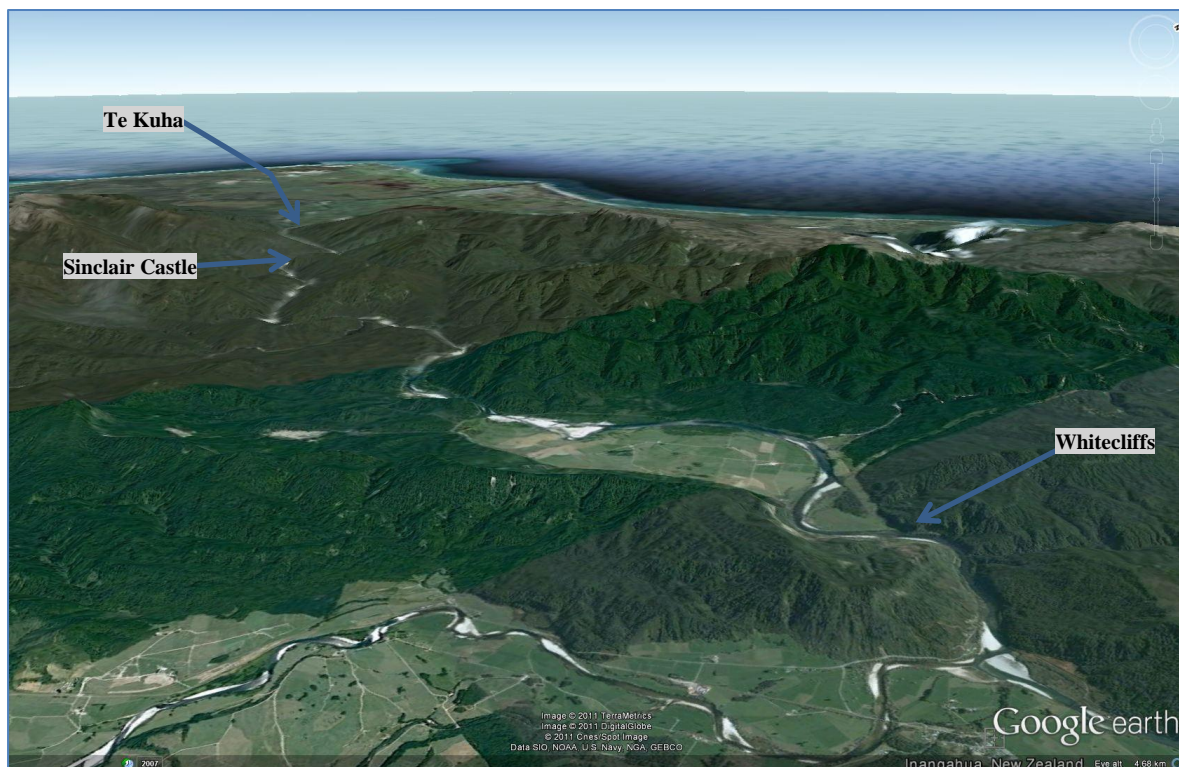
The review of aerial photography from 1947, 1959 and 1985 has identified a number of comparatively large-scale, pre-1947 landslides that the rail corridor was formed through. Most notable is the failure surface at Sinclair Castle, around SNL119km (Figure A3.2.2, Inset K). Comparatively more recent failure surfaces can be seen in the aerial photography, including the highlighted features in Figures A3.2.1 and A3.2.2. The rate of re-vegetation limits these typically smaller-scale landslides, in terms of date of occurrence, to closely preceding the date the photograph was flown (on the scale of years). Accordingly, detail incorporated into the landslide inventory presented in Chapter 3 from the aerial photograph review has a slight bias towards the dates the photographs were flown.

### A3.2.2 Google Earth and MapToaster Topo New Zealand

Additional aerial imagery is available from Google Earth and MapToaster Topo New Zealand. Full colour photography available from Google Earth for the project area is summarised below (SNL metrage and date flown):

- 96 – 99km: 5 September 2007
- 99 – 107km: 18 January 2008
- 107 – 126km: 4 January 2003

The colour changes in Figure A3.2.5 denote the different aerial flight dates outlined above. The quality of the Google Earth images is variable. Large topographic features can be viewed but a detailed review is not feasible in the absence of stereo coverage.



**Figure A3.2.5: Oblique photograph showing aerial imagery available from Google Earth**



No slope movement features are visible on Google Earth associated with Whitecliffs. The 1987 landslide at Windy Point is visible. Other features include the landslide at SNL108.60km seen in 1985 photographs (Figure A3.2.1, Inset D) is still visible in Google Earth (2003), and recent slope movement west of Tunnel 6, near SNL124.40km. The images from 2008 are poor quality.

Aerial images available from MapToaster Topo New Zealand were flown in 2000/2001, and cover the Lower Buller Gorge from near SNL111km to 126km only. Image quality is average, as seen in Figure A3.2.6, which shows the only recent slope movement feature immediately adjacent to the rail alignment at the same location as in Google Earth (2003) at ~SNL124.40km. This feature is not visible in 1985 (refer Figure A3.2.4).



**Figure A3.2.6: Slope movement in 2000/2001 near SNL124.40km. Tunnel 6 is visible on right-hand side of aerial photograph.**

### A3.3 1929 Buller earthquake

The rail corridor was not completely formed in 1929 so this section outlines any detail available regarding landslide occurrence in the general region. The 1929 Buller earthquake has been well researched. Literature relevant to this thesis includes: Henderson (1937); Hancox et al (2002); and Dowrick (1994). In addition, the Buller District Council commissioned a lifelines study that was reported in 2006 (Buller District Council, 2006). This study adopted an Alpine Fault Earthquake scenario and incorporated impacts from both the 1929 and 1968 earthquakes. This section is based predominantly on information from this study and the other sources listed above.

The Buller earthquake occurred on 17 June 1929 and measured  $M_s 7.8$ . The epicentre was located approximately 15km northwest of Murchison in a remote and sparsely populated area. Vertical displacement of 4.5m occurred on the White Creek Fault approximately 11km west of Murchison. Horizontal displacement was measured up to ~2.5m. Due to the steep and heavily vegetated environment around the epicentre the measured surface rupture only extends for 8km, in an approximate north to south alignment (Figure A3.3.1), but it is estimated that the total rupture length was in the order of 30-50km (Dowrick, 1994). The style of failure was predominantly reverse faulting.

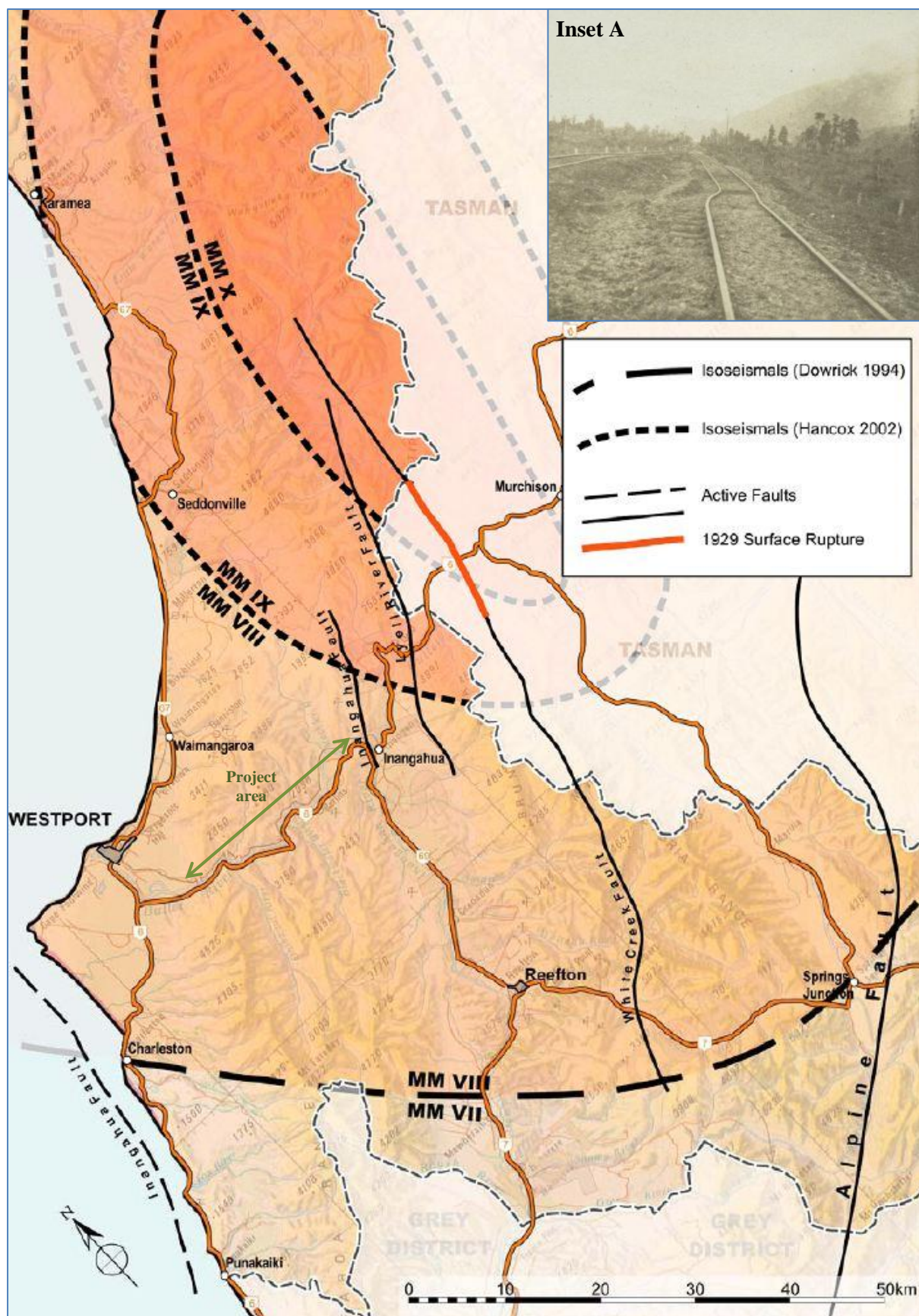


Figure A3.3.1: Modified Mercalli Intensities from the 1929 Buller earthquake (from Buller District Council, 2006). Project area location indicated by arrow. Inset A shows an example of damage to rail near Murchison (sourced from: <http://mp.natlib.govt.nz/detail/?id=40138>).

Westport had a population of around 4,000 in 1929. Impacts to key lifelines in Westport are summarised by Dowrick (1994), and include loss of electricity for 13 hours; water supply lost for nine days; and, the telephone exchange was unavailable for three days. Road and rail links were significantly impacted due to landslides, as discussed in the following section.

### **A3.3.1 Earthquake-generated landslides**

Modified Mercalli Intensities (MMI) experienced in the Buller Region are shown on Figure A3.3.1, which indicates a shaking intensity of MMVIII in the area between Westport and Inangahua. The generation of landslides is an almost certain consequence for this shaking intensity level. Seventeen fatalities resulted from the 1929 earthquake, of which 14 were due to landslides, two due to rockfalls in mines and one indirect death due to transportation delays in receiving medication (Dowrick, 1994).

The railway line through the Lower Buller Gorge was not fully constructed in 1929 and there is no detail available on damages sustained on the section that was formed near Te Kuha. Damage to bridges and spreading of embankments were the predominant issues noted (Buller District Council, 2006). Rail services were functional again from 24 June 1929 (seven days after the earthquake). In comparison, the road network took a considerably longer time to reinstate, including 22 months in the Upper Buller Gorge.

A comment was made in regard to SH6 at Whitecliffs that required a temporary detour due to rockfalls in 1929 (Buller District Council, 2006). Discussion in the Buller District Council (2006) report indicated that the comparatively low damage sustained to the rail network was due to the majority of railways being located away from steep slopes and the associated landslide-prone areas. A similar event today could result in many weeks of disruption as the Lower Buller Gorge due to landslides and rockfalls (Buller District Council, 2006). Aftershocks and rainfall were also noted as causing continued delays to repair work in 1929 as more landslides were generated or reactivated.

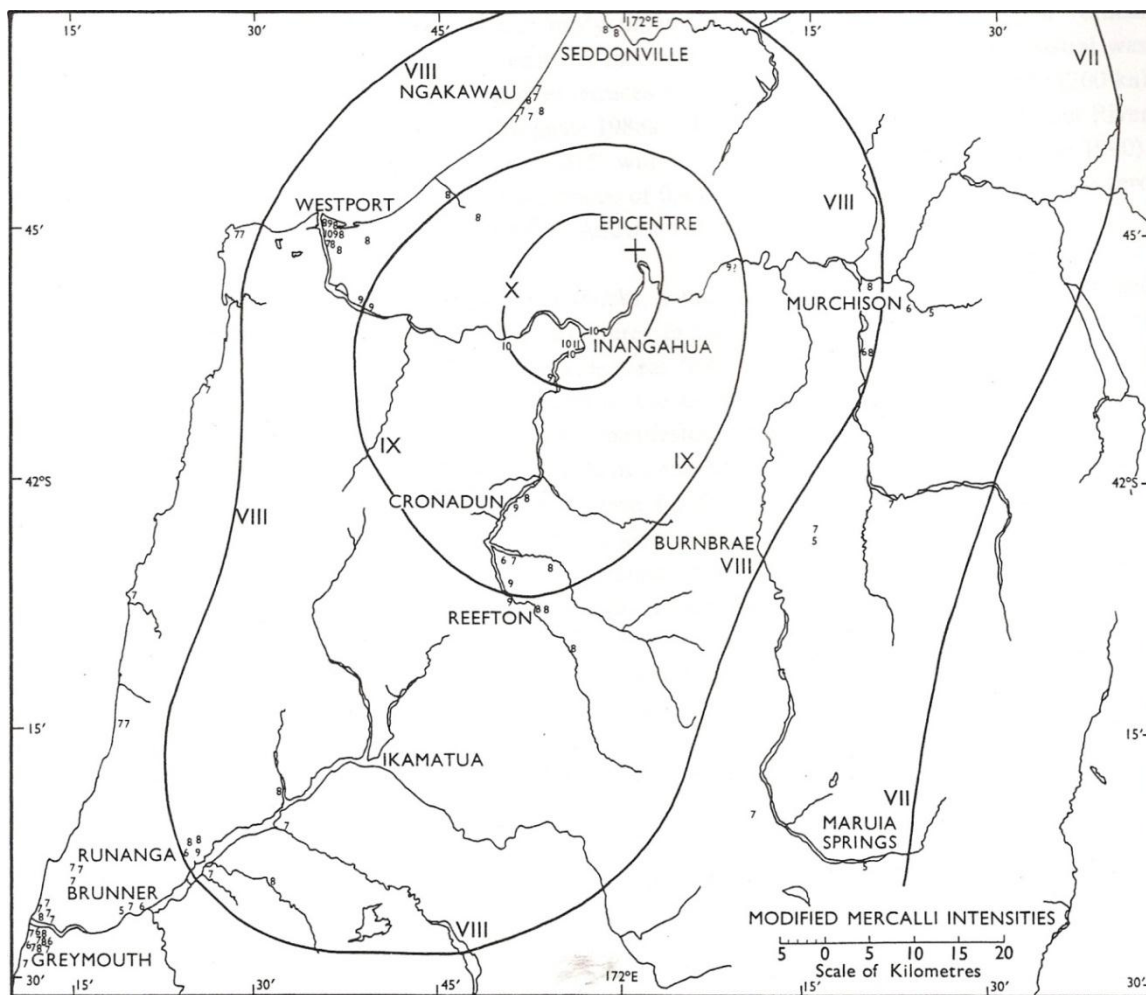
## **A3.4 1968 Inangahua earthquake**

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A literature review identified a large amount of information available regarding impacts and research related to the 1968 Inangahua earthquake. Key publications reviewed include Anderson et al (1994); Adams et al (1968); and, Adams and Lowry (1971). Invaluable information was also obtained from the Inangahua Earthquake Museum. Available aerial photographs flown in 1970 were reviewed to determine any visible, large-scale landslides that directly impacted the rail corridor. This section summarises relevant information obtained from the above sources, focussing on damages sustained to the road and rail corridor as a result of earthquake-generated landslides.

### A3.4.1 Background

The magnitude 7.1 Inangahua earthquake occurred on 23 May 1968 at 1724 (UT). The local time was 0524 on 24 May 1968 (Downes, 1995). The epicentre was approximately 15km north of Inangahua, as shown on Figure A3.4.1 (defined by Anderson et al, 1994). Greater than 800 aftershocks were recorded within six weeks of the main event, including 12 with magnitudes greater than 5 (Adams and Lowry, 1971). Three fatalities occurred, including two related to the collapse of limestone cliffs at Whitecliffs, adjacent to SH6. The shaking intensity caused railway lines to twist, buckle, derailed two trains, and resulted in approximately 100km of tracks that had to be replaced (Buller District Council, 2006).



**Figure A3.4.1: Modified Mercalli Intensities from the 1968 Inangahua earthquake (based on Anderson et al, 1994).**

The MM shaking intensities shown on Figure 3.4.1 indicate the project area received MMX at the eastern end, down to MMVIII near Te Kuha. The delineation of MMX areas is similar between the two isoseismal maps (Figures A3.4.1 and A3.4.2). Anderson et al (1994) shows a slightly larger area that experienced MMIX compared to Buller District Council (2006) who derived their isoseismals from Hancox (2002) and Dowrick et al (2003).



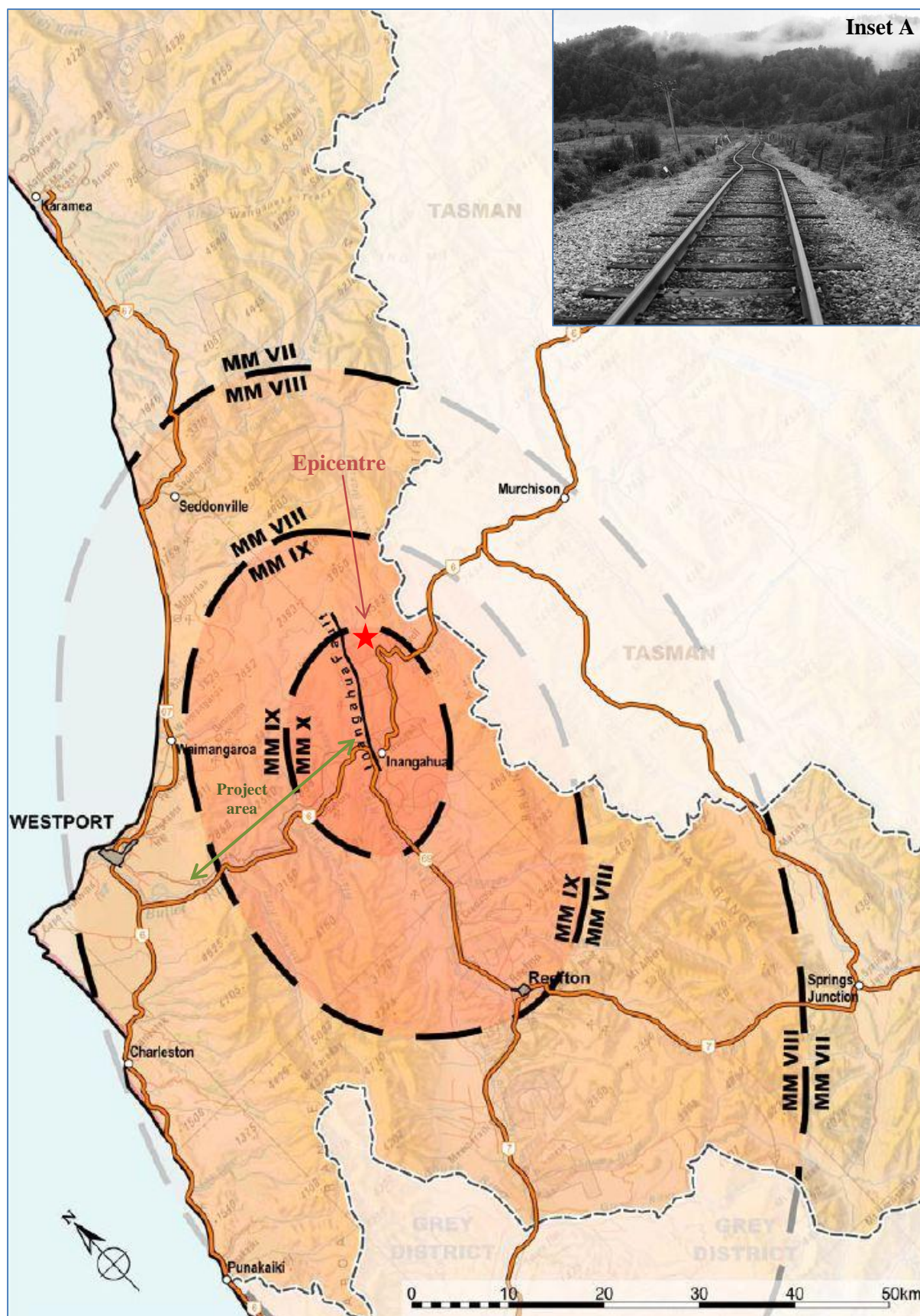


Figure A3.4.2: Modified Mercalli Intensities from the 1968 Inangahua earthquake (based on Buller District Council, 2006). Project area location indicated by arrow. Inset A shows an example of the damage sustained to rail near Inangahua (sourced from: <http://www.geonet.org.nz/earthquake/historic-earthquakes/top-nz/gallery.html>).



### A3.4.2 Inangahua Earthquake Museum

The Inangahua Earthquake Museum contains a variety of information including eye-witness accounts of the 1929 and 1968 events, flooding events, newspaper articles, figures, photograph displays and numerous technical reports. Impacts to the rail network were most concisely reported in a letter provided to the earthquake museum regarding restoration work, and a detailed work log, by JC & P Fitzpatrick Limited, dated 30 March 2007 (Fitzpatrick, 2007). The following discussion is based on this document and any direct quote are italicised.

The SNL was opened 40 days after the main, 24 May 1968, event and was recorded as ‘*wrecked and unusable*’ for 20 miles (~32km) either side of Inangahua. It took five days to complete the initial assessment of damage to the rail corridor, which included over 40 landslides, collapsed embankments and one major bridge to repair.

When reopened, speeds were restricted to 15 miles per hour (mph) (~24km/hour) with some areas reduced to 6mph (~10km/hour). To increase train speeds to 30mph (~48km/hour), 100 miles (~160km) of track needed re-ballasting and the equivalent of 40 additional landslides remediated. Landslides encountered during the restoration work were described as varying from ‘*extensive cutting collapses*’ to ‘*rock outcrops which had simply fallen from a considerable height onto the track shearing the steel railway like a guillotine*’.

Relevant contents of this letter report, in terms of landslide occurrence, are summarised in Table A3.4.1. Locations in the letter reported were in units of miles, these have been converted to current SNL metrage locations (km). Reference was made to the 61 mile peg as being located ‘*under the cliffs west of Inangahua*’. This distance correlates to SNL98.170km, near the western extent of Whitecliffs.

**Table A3.4.1: Landslide occurrence in the Lower Buller Gorge based on restoration works completed after the 1968 Inangahua earthquake (based on Fitzpatrick, 2007)**

Date (1968)	Detail (direct quotes in italics)	Location	
		Miles	SNL (km)
26 May	<i>Travelled to first slip and then walked to 69m peg recording location and size of slips.</i>	69m	111.044
31 May	Blasting a section of rock (or rocks) that had fallen at 73m was unsuccessful. Also a large block at 72m70c that required blasting. Slip at 72m40c was larger than previously assumed. Issues around Tunnel 3.	73m	117.482
		72m70c	117.280
		72m40c	116.677
2 June	More movement observed at a slip at 72m78c.	72m78c	117.441

Date (1968)	Detail (direct quotes in <i>italics</i> )	Location	
		Miles	SNL (km)
3-6 June	Slip at 73m referred to on numerous occasions, more movement observed on 4 June. New slip at same location on 5 June. Comment made ' <i>trolleyed through to Whitecliffs</i> ' on 5 June – no mention of any landslides at this stage. Slip at 73m cleared on 6 June.	73m	117.482
7 June	Crib log wall was constructed to stabilise toe of slip at 73m. Minor slip at 76m6c.	76m6c	122.430
8 June	First slip at Whitecliffs clear. No further detail provided or precise location. 68m culvert was noted as requiring clearance. 6mph restriction applied between Tiroroa and Rahui.	68m	109.435
9-28 June	The restoration work logs provided detail regarding the dispatch of machinery but no specific locations or works involved were noted between 9 and 28 June.	NA	NA
29 June	Slips noted at 69m and 59m50c. No further details provided.	69m 59m50c	111.044 95.957

A newspaper article referred to rail damage to Tiroroa, which likely correlates to the main landslide referred to in Table A3.4.1 at SNL117.482km. The newspaper source and date are unknown, and no specific details were provided. A Nelson Evening Mail article, dated 25 May 1968, refers to landslides at Windy Point, which is near the original location of Tunnel 7 in the Lower Buller Gorge (daylighted in 1987 due to landslide damage). Significant loss of ballast at Rahui was also noted.

The locations referred to in Table A3.4.1 are limited in terms of a full inventory for landslide occurrence, but do provide confirmation that blocks were displaced at Whitecliffs that impacted the rail. Attempts were made to contact the author of the letter directly but no response was received.

#### A3.4.3 1970 aerial photography

To determine whether there were any additional direct impacts of large-scale earthquake-generated landslides to the rail corridor after the 1968 Inangahua earthquake, aerial photography flown in 1970 was ordered from Aerial Surveys Limited (ASL). The images obtained do not cover the entire project area, but the east-west run does incorporate Whitecliffs and other locations of the rail corridor as far west as SNL122km. A total of 15 contact prints were viewed at a scale of 1:15,000 (photograph reference numbers 37046 to 37061).

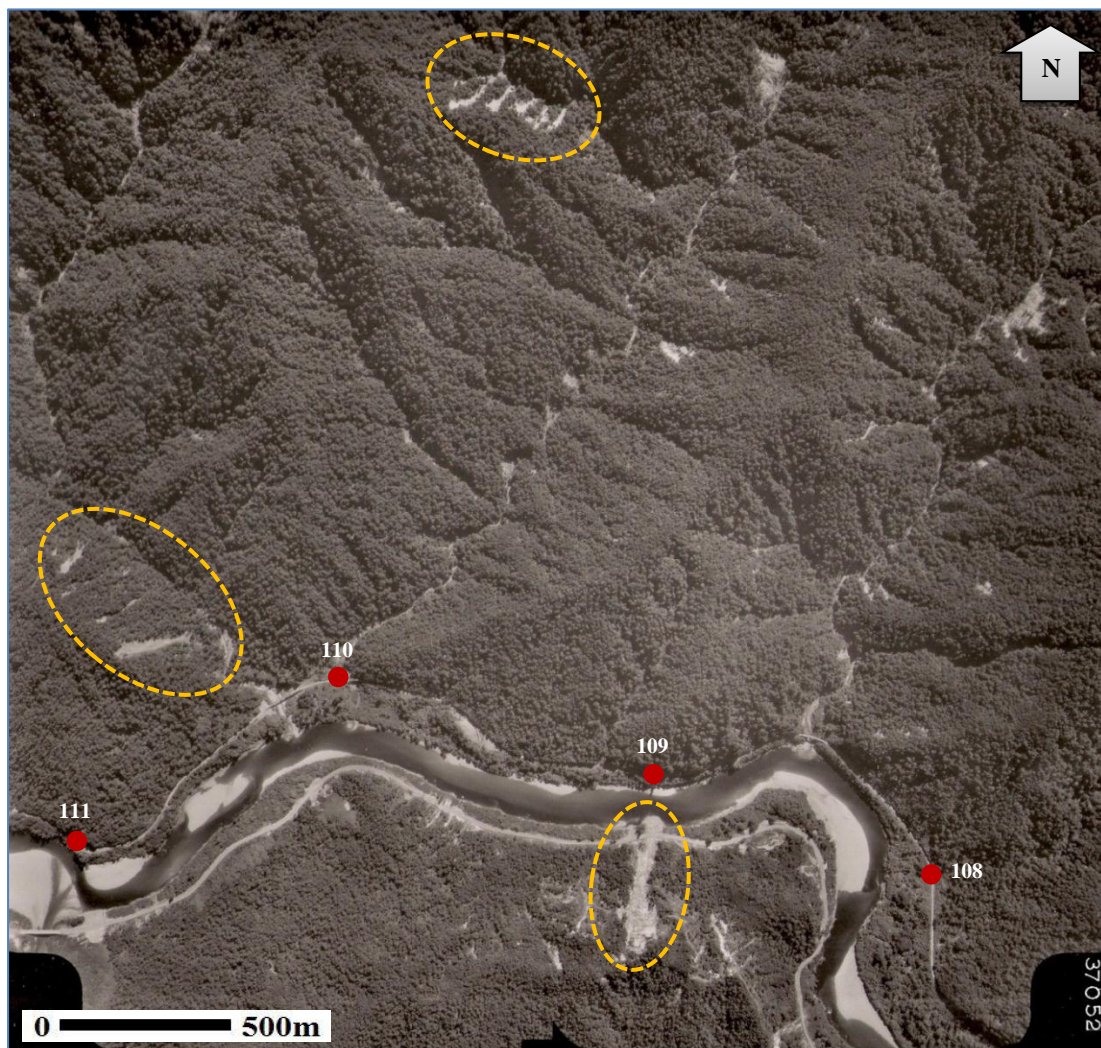
There were no direct landslide impacts to the rail corridor evident in the photographs viewed from 1970 (approximately 15 months after the main earthquake event) but fresh scars in the landscape were present north of the rail alignment. Three photographs are shown in Figures A3.4.3 to A3.4.5 that provide an example of landslides that were most likely generated by seismic shaking north of Whitecliffs (Figure A3.4.3); in the vicinity of SNL108km to 111km (Figure A3.4.4); and, on the northern facing slopes north of Sinclair Castle (Figure A3.4.5). There were consistently more slope movement features visible on the southern side of the Buller River in all photographs viewed, which highlights the vulnerability of SH6 in particular to a future seismic event of similar or greater magnitude.



**Figure A3.4.3: 1970 aerial photograph in the vicinity of Whitecliffs and SNL97km to 99km.**



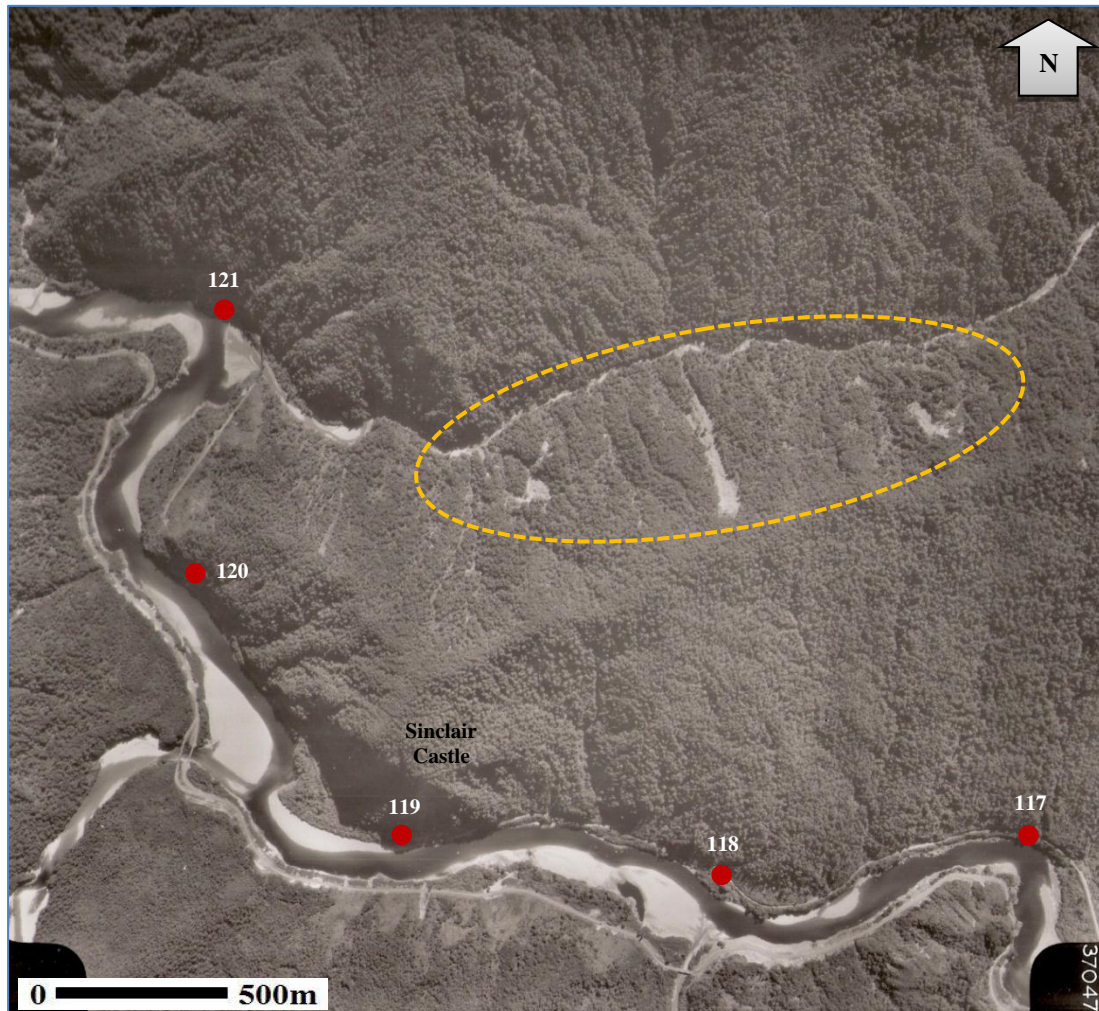
The detached block at SNL98km shown on Figure A3.1.1 from 1985 (Inset B) is also visible in the 1970 photograph (directly above the SNL98km location on Figure A3.4.3). No other slope movement features are visible adjacent to the rail corridor at Whitecliffs. In comparison, widespread landslides and rockfalls are clearly visible on the southern side of the Buller River, adjacent to SH6 and the slopes above. The circled area on Figure A3.4.3 highlights recent slope movement associated with the western side of the hillslope north of SNL98km.



**Figure A3.4.4: 1970 aerial photograph in the vicinity of SNL108km to 111km.**

A large landslide is visible on Figure A3.4.4 that crossed SH6 (southernmost circled location). There are no large-scale slope movement features visible that directly intersect the rail corridor in this area but landslides are visible to the north. Circled locations north of the Buller River show landslides associated with the north to northeastern sides of the two highlighted hillslopes. Comparatively smaller fresh scars in the landscape, than those highlighted on Figure A3.4.4, are also present on both sides of the Buller River but particularly associated with SH6.

Sinclair Castle is shown on Figure A3.4.5. There are no visible slope movement features associated with the steep topography at Sinclair Castle but the shadow in this area may obscure comparatively smaller scale failures. There are also no direct impacts from landslides visible between SNL117km and 121km. The highlighted area shows fresh scars in the landscape associated with the northern facing slope north of Sinclair Castle.



**Figure A3.4.5: 1970 aerial photograph in the vicinity of Sinclair Castle and SNL117km to 121km.**

#### **A3.4.5 New Zealand Society for Earthquake Engineering, 1968**

Duckworth (1968) described a number of landslides that he observed had impacted the rail corridor. The volume of material required to be removed due to landslides in the rail corridor (including the SNL west of the project area) was around 30,000 cubic yards ( $\sim 23,000\text{m}^3$ ). General comments made by Duckworth (1968) regarding rail construction included that construction batters accentuated the pre-existing steepness of the slopes that the rail was attached to, and no benching was undertaken during the original formation of the corridor. The implication of these construction practices is that a thin (typically 1-3m thick) colluvium unit remained in place and had the potential to regress upslope.



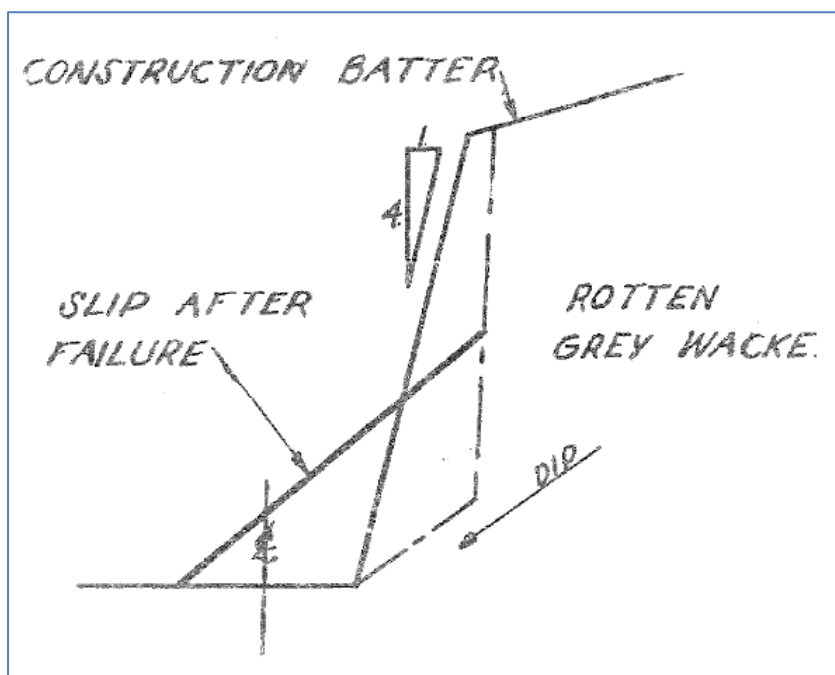
An annual rainfall of 164 inches (>4,000mm) was referred to that created additional challenges in the catchments above the rail corridor due to weakening of soil surrounding loose boulders up to 10 long tons (or 10 tonnes) that could be released during periods of long duration and/or high intensity rainfall events. Landslides witnessed by Duckworth (1968) included:

**A. Limestone Bluffs (Whitecliffs at 60m58ch or 97.737km):**

According to Duckworth (1968), prior to the Inangahua earthquake the cliff face profile at Whitecliffs consisted of a near vertical face. The earthquake resulted in the appearance of vertical cracks and large blocks were dislodged onto the ground below (some >100 tonnes). This is the first and only reference to a major rockfall that has been reported for Whitecliffs due to seismic shaking during the Inangahua earthquake. Fitzpatrick (2007) refers to ‘slips’ at Whitecliffs that impacted the rail corridor in 1968 but no detail was provided to indicate that the failures represented large-scale instability or cliff-face damage.

**B. Mudstone (67m40ch or 108.630km):**

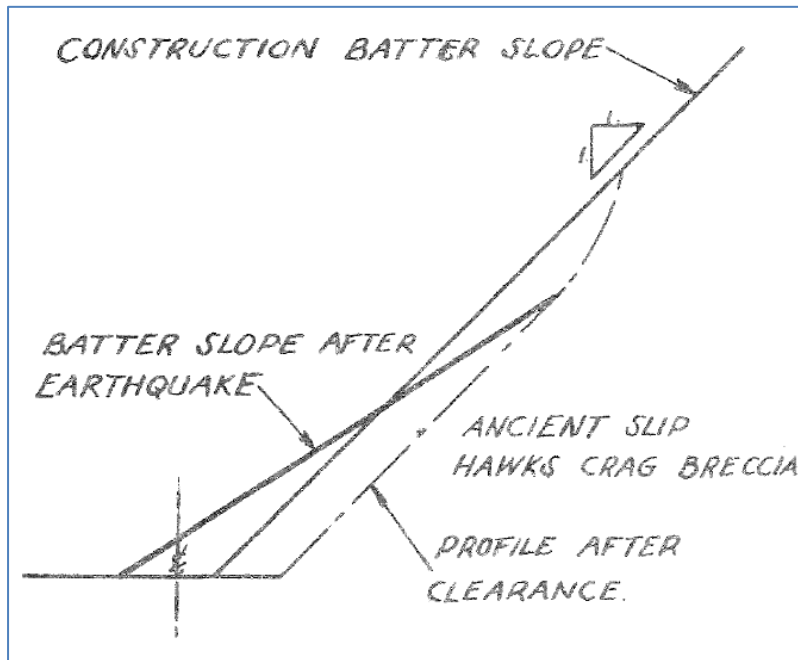
At 67m40ch (108.630km) Duckworth (1968) observed rock slides that were predominantly comprised of mudstone, and described as ‘large shattered rock’. Batters at this locality were near-vertical, as indicated by Figure A3.4.6 from Duckworth (1968). There is a discrepancy between the labelling of Figure A3.4.6 as ‘greywacke’ and the text description of ‘mudstone’. The geological unit at this location is mapped by Bowen (1964) as Greenland Group (Figure 2.3, Chapter 2).



**Figure A3.4.6: Sketch diagram from Duckworth (1968) showing a landslide at SNL108.630km**

**C. Hawks Crag Breccia (73m peg or 117.482km):**

Landslide material at this location consisted of massive, hard conglomerate (granite and greywacke within a sand/silt matrix) with boulder sizes up to 50 tonnes. The landslide occurred in an area that had failed historically, and was reactivated by the Inangahua earthquake. Batter slopes were indicated by Duckworth (1968) at approximately 1:1 (Figure A3.4.7).



**Figure A3.4.7: Sketch diagram from Duckworth (1968) detailing a landslide at SNL117.482km.**

Remedial works at this location encountered major issues when removal of the rock slide toe support triggered additional slope movement, exposing a progressively wider and higher failure surface. A ‘crib log’ retaining structure was constructed to stabilise the slope in this area. This event correlates to the references made by Fitzpatrick (2007) around the same location (refer Table A3.4.1).

#### **A3.4.6 New Zealand Society for Earthquake Engineering, 1969**

The New Zealand Society for Earthquake Engineering collated a report titled ‘A Preliminary Report on the Inangahua Earthquake New Zealand, May 24, 1968’ in January 1969, which provides some detail specific to landslide types observed in the region. Landslide distribution was predominantly in a 10 mile radius (~16km) of Inangahua with noticeably fewer to north and northwest. Mechanisms of slope failure within the region included (Lensen and Suggate, 1969; Falconer and Lenson, 1969):

- Rockfalls in Tertiary limestone (particularly at Whitecliffs, adjacent to SH6).
- Slides within Upper Tertiary sediments (released on bedding planes).
- Defect-controlled rock slides (on joint surfaces) were reported as common in granite.

- Slides originating from very steep slopes that consisted of blocky/weathered surficial material.
- Numerous debris slides within near-surface material at weak crush zones in granite.
- Earthflows of weathered muddy sandstone Upper Tertiary material, particularly at Inangahua.
- Low number of rotational slides in Upper Tertiary sediments (mostly mudstone).

Major Inangahua Earthquake damage to the road network within the Lower Buller Gorge was reported by Douglas (1969) as extending from Inangahua to Windy Point, which comprises almost the entire gorge. Douglas (1969) also provided an estimate of the total assessed volume of landslide material that had to be removed from all road networks in the region after the Inangahua earthquake as exceeding 400,000 cubic yards ( $>306,000\text{m}^3$ ).

#### **A3.4.7 New Zealand Ministry of Civil Defence, 1970**

The New Zealand Ministry of Civil Defence (NZMCD) collated an 87 page report on the Inangahua Earthquake in 1970 (NZMCD, 1970). The rail corridor in the Lower Buller Gorge was referred to briefly but no specific details on landslides were recorded. Approximately 40 miles (~65km) of rail became twisted due to ballast movement, which corresponds with the letter report discussed in Section 3.8.2. Reference was made to damage to combined road-rail bridge approaches (outside of the project area) at Inangahua, Waitahu, Boatmans and Landing.

#### **A3.4.8 University of Canterbury, 1970**

Shepherd et al (1970) noted that by the time an inspection of the rail line was conducted by a University of Canterbury survey team in August 1968, the line was reopened and any direct impacts from landslides to the rail itself were obscured. No comments were made in regards to the surface features related to slope movement aside from mention of four ‘slip surfaces’ on the railway side of the Lower Buller Gorge. These four slips were measured and plotted, together with 14 slips adjacent to SH6, to show the slope of every slip with respect to slope orientation.

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# Appendix 5.1

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## *Case studies for three key risk sites (107.5, 118.5 and 121.0km)*

Three key risk sites were identified from the historic record of landslide occurrences in the project area, and these were inspected during numerous site visits between 2009 and 2011. It is recognised that the combination of steep terrain and high rainfall result in many sites within the project area that are prone to episodic slope failures, and the risk of substantial impacts from future earthquakes remains an issue in the management of operations. The specific sites discussed in Appendix 5.1, which displayed large to very large ( $>1,000\text{m}^3$ ) and/or relatively frequent ( $\leq 2$ -year) instability, are detailed below:

- **SNL107.5km** – bedding-controlled failures have occurred in Kaiata Formation sandstones and mudstones that dip towards the railway line;
- **Sinclair Castle (SNL118.5km)** – large but now inactive pre-historic landslide is evident on the northern bank of the Buller River. This area is subject to episodic debris slides and flows originating in granitic source areas above steep gullies, particularly associated with the eastern side of the original pre-historic failure; and
- **Cascade (SNL121.0km)** – frequent (~once every 2 years) debris slides and flows that originate in steep catchments, and sourced from granite, above the rail corridor. The resultant impacts have required on-going remedial works and monitoring by KiwiRail.

The following approach has been adopted in assessing the landslide susceptibility for each site, with the primary purpose being to identify the geological and topographic controls on landscape development that influence slope instability and the potential effects on rail operations:

- Geological, geomorphological and hydrological setting;
- Past and/or present slope instability impacting rail operations;
- Remedial measures undertaken to improve line security; and
- Future management issues and recommended practices.

The three sites represent on-going slope stability challenges to rail operations. Landslide risk management is a tool that will enable KiwiRail to assess the slope stability risk further, with a consistent approach, past this initial risk analysis stage.

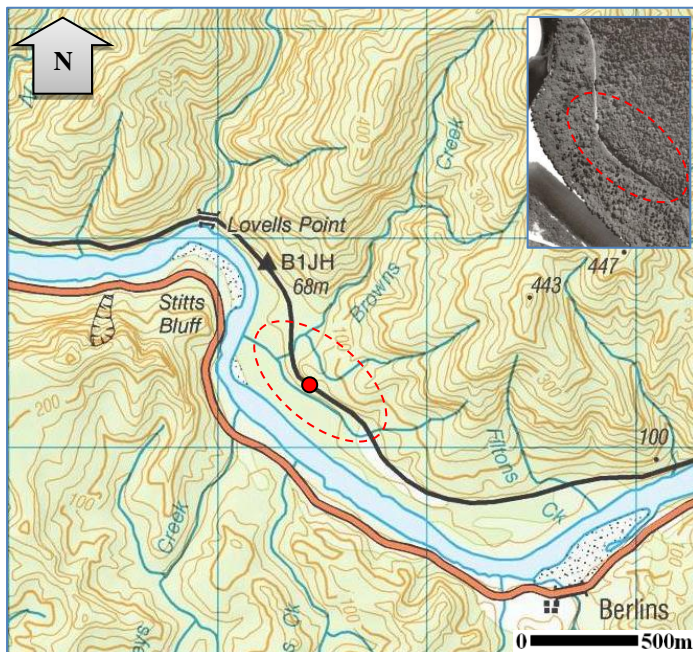
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## CASE STUDY 1: Bedded Kaiata Formation at SNL107.50km

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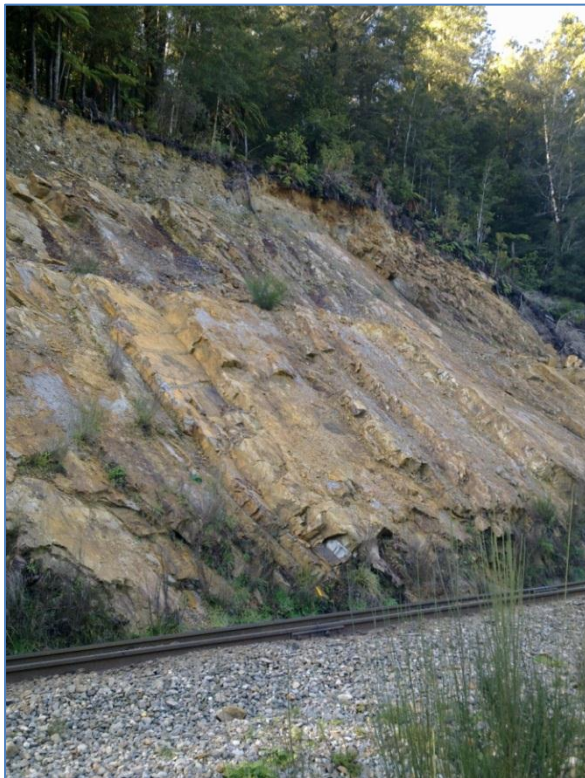
Landslides have resulted in impacts to rail operations in the vicinity of SNL107.50km. A derailment occurred in 2004 within the circled red area on Figure A5.1. No information was made available regarding details of the landslide. The geological map shown in Chapter 4, Figure 4.3, indicates that bedrock in the vicinity of SNL107.50km comprises Hawks Crag Breccia, with Greenland Group outcrops to the north (based on the mapping of Nathan et al, 2002). As seen in Figures A5.2 to A5.3, the unit at this location is actually Kaiata Formation mudstone and sandstone. This is indicative of the limitations of the geological mapping available and the comparatively detailed nature of the rail corridor that is being evaluated in this thesis. The nearest mapped Kaiata Formation exposure is approximately 2km from SNL107.50 (Chapter 4, Figure 4.3).

Remedial works after the 2004 landslide comprised benching the slope and removal of loose rock along an approximate length of 150m adjacent to the rail, as seen in Figures A5.2, A5.4 and A5.5. The photograph of the coal train in Figure A5.5 shows the close proximity of the cut slope to the railway. The bench surface is near level, and loose rock observed in the catch area at the base of the slope indicates the rock trajectory is at times sufficient to impact the rail corridor. The distance between the edge of rail and base of the cut slope averages less than 2m, and the surrounding area is heavily vegetated. It is possible that the presence of Kaiata Formation may not have been recognised until the 2004 slope failure that impacted the rail operations.

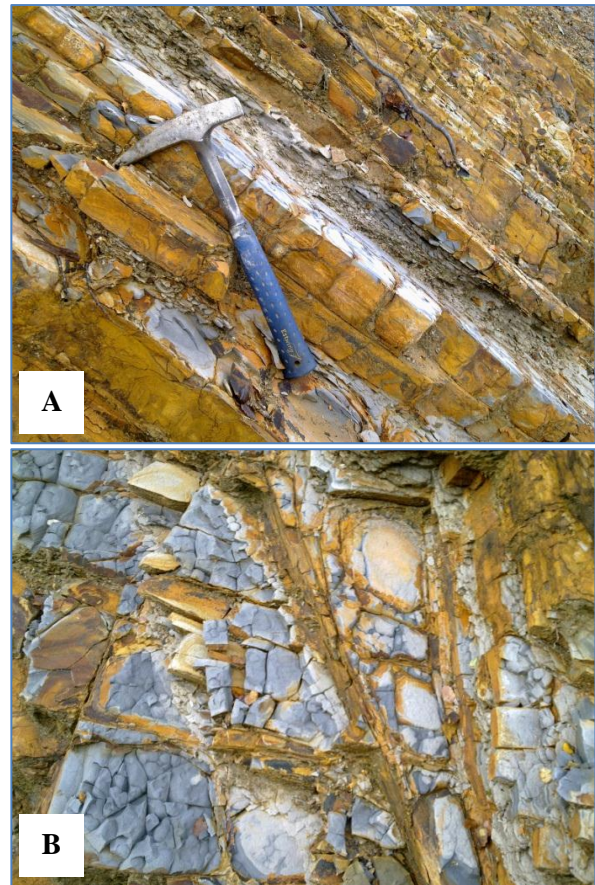


**Figure A5.1: Location and topography in vicinity of SNL107.50km (half metrage mark indicated by red circle). Inset shows an aerial view from 1985.**





**Figure A5.2: View of rock outcrop near SNL107.50km. Bench tapers out on the eastern (right-hand) side of photograph.**



**Figure A5.3: Close-up view of Kaiata Formation interbedded sandstone (A) and mudstone and minor shear zone in batter (B).**



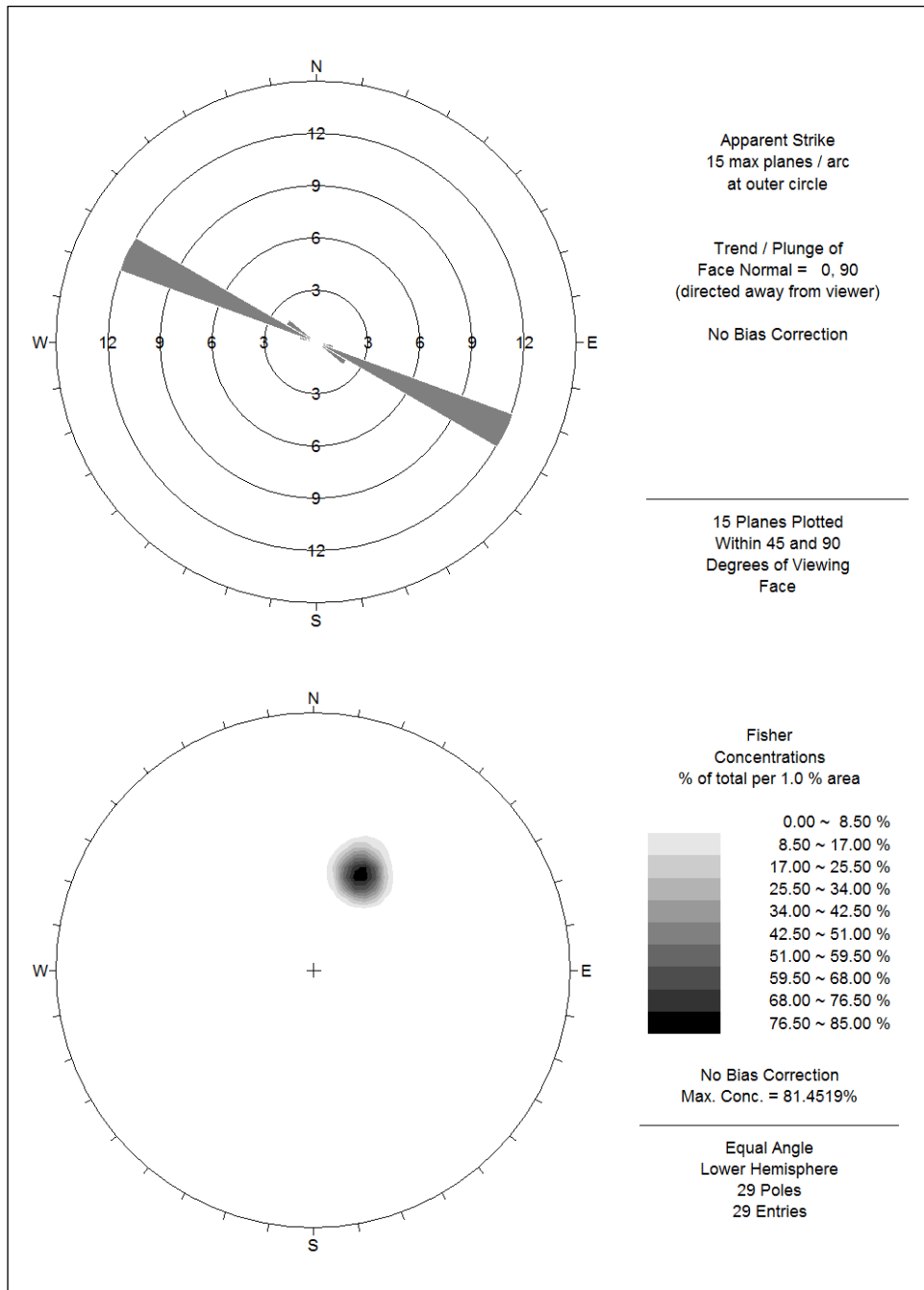
**Figure A5.4: Base of slope (view looking south-east) with bedding daylighting out of slope, dipping around 40-45°.**



**Figure A5.5: Scale of batter and proximity to coal train movements. Note the limited visibility around the track curve.**

## Rock material and rock mass characteristics

Figures A5.2 to A5.5 show bedding dipping around  $45^\circ$  and striking sub-parallel to the railway ( $295 \pm 10^\circ$  True) for a distance of approximately 100m. Measured strikes varied between  $184$  and  $305^\circ$ , and dips between  $39$  and  $53^\circ$  to the south, as depicted by the close concentration of 29 poles to bedding in Figure A5.6.



**Figure A5.6: Bedding orientation measured in Kaiata Formation at SNL107.50km**

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Bed thickness measured along the exposed face ranged between 150 and 400mm at the western end, and up to ~500mm at the eastern end of the cutting. The sandstone interbeds are thicker than the mudstones and siltstones, which are more closely laminated. Following the New Zealand Geotechnical Society (2005) the Kaiata Formation rock material can be described as: “*greyish-brown, interbedded sandstone, siltstone and mudstone; slightly to moderately weathered; moderately thin to moderately coarsely bedded; weak to moderately strong*”.

The principal rock mass defect is bedding, which is moderately widely spaced, and persistent for greater than 10m in the cut face. Joint sets normal and parallel to the strike of bedding are evident, and are also persistent for more than 10m. The joint sets are variably spaced from 0.1m to greater than 1.0m. Minor shear zones cross-cutting bedding were recognised (Figure A5.3B), but no significant offset of bedding was noted. Shear zones observed will act as localised release surfaces for relatively small bedding-controlled slabs (up to about 2m<sup>3</sup> in volume).

In addition to bedding- and joint-release of blocks, the Kaiata Formation mudstones typically undergo slaking from wet-dry cycles. This is evident in the finer-grained units within the rock mass, and blocks of variable dimensions up to about 0.5m<sup>3</sup> in volume accumulate on the intermediate bench and adjacent to the rail. Over time, fretting from the bedded rock mass can be expected to take place due to a combination of slaking and slabbing, and regression of the steeper cut batter in overlying gravels is also anticipated.

#### **Site hydrology and hydrogeology**

As seen in Figure A5.1, a tributary to Browns Creek flows irregularly down the slope above SNL107.50km. This stream is located in the bush above the cut face. Seepage may occur onto the formed batter face. Access limitations and time constraints working within the rail corridor prevented a detailed inspection of this water feature. Intact Kaiata Formation rock material has a very low hydraulic conductivity but fracture-controlled seepage flows would enable some water infiltration via the surficial gravel unit into the underlying bedrock (Wyllie and Mah, 2004). Progressive reduction of the intact rock strength due to slaking and slabbing could be expected as a result of water infiltration (Bell and Pettinga, 1983).

Trees at the top of the remediated section will also over time fail due to intense or prolonged rainfall event, and the catch berm is expected to become filled with debris and/or failed vegetation in the longer term ( $\geq 5$  years). Some debris can be expected to periodically reach the rail level, in spite of the measures undertaken to stabilise the slope. Control of stormwater, and monitoring of seepage by periodic inspection, are considered necessary for this site where measures have been implemented already to improve overall stability along a relatively short section of track.

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## Present and future stability

The measures undertaken since the 2004 slope failure at 107.50km were required because the bedding strike is almost parallel to the rail alignment. The slope has been cut parallel to the ~45° dip of the Kaiata Formation bedding, with a single bench around 7-8m above the rail level and a similar height cut above this to the tree line in gravels. To the west the rail curves back to the north and cuts the bedding obliquely, enhancing overall stability. In terms of Landslide Risk Management, the following matters are considered important for understanding the hazard and minimising the risk of future impacts from slope failures at this locality:

- Over time the Kaiata Formation bedrock will undergo slaking, and slabbing will occur from relatively thin (<1m thick) small volume (typically less than 2m<sup>3</sup>) blocks released on bedding. This material will accumulate on the bench, and at rail level.
- Localised collapse of the weathered outwash gravels at the tree line can be expected to occur over time, which will progressively fill the intermediate bench with debris. Trees will be undercut, and may at times reach track level during high intensity or long duration rainfall events.
- Regular inspections of the site could be carried out to ensure that the catch berm has adequate capacity, and that stormwater drainage has not been compromised by debris accumulation or vegetation collapse.
- Extending the bench another 50m to the east to improve the ability to intercept debris above track level would be beneficial but it is recognised the cost for these works may not be considered economic. Consideration could also be given to a second bench at the level of the gravels to improve long-term stability.

Remedial works completed at SNL107.50km have reduced the potential impacts from future landslide events. Increasing the berm angle of the benched area, back into the slope, would provide a more efficient catch area for slabs of rock and the overlying gravel and vegetation to be captured in the future. Increasing the bench towards the east would also increase efficiency but the volume of rock required to be moved may prevent this as a feasible mitigation option. Routine monitoring and maintenance of this area will minimise the build-up of source material that could mobilise during high intensity or long duration rainfall events.

Long-term mitigation measures at SNL107.50km are a matter for KiwiRail, and the options identified here are provided as part of an evaluation of the effectiveness of the works undertaken to date within the framework of Landslide Risk Management.



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## CASE STUDY 2: SNL118.50km – Sinclair Castle

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Sinclair Castle is repeatedly referred to within this thesis and in discussions with any person having background knowledge of the project area. The location and elevation of Sinclair Castle is shown on Figure A5.7, including the geomorphic expression of what is interpreted as a pre-historic, re-vegetated landslide that impacted the course of the Buller River. The rail has been formed through the toe of the pre-1947 landslide, this being the earliest dated aerial photography available on which the much older landslide is evident. Gully-controlled smaller slope instability features on the older, and seemingly stable, landslide are of concern to railway operations. These are evident through the trees within the “Pre-1947 Landslide Area”, as shown in Figure A5.7, and they episodically reactivate during intense rainstorms to release rock debris towards or onto the rail corridor.



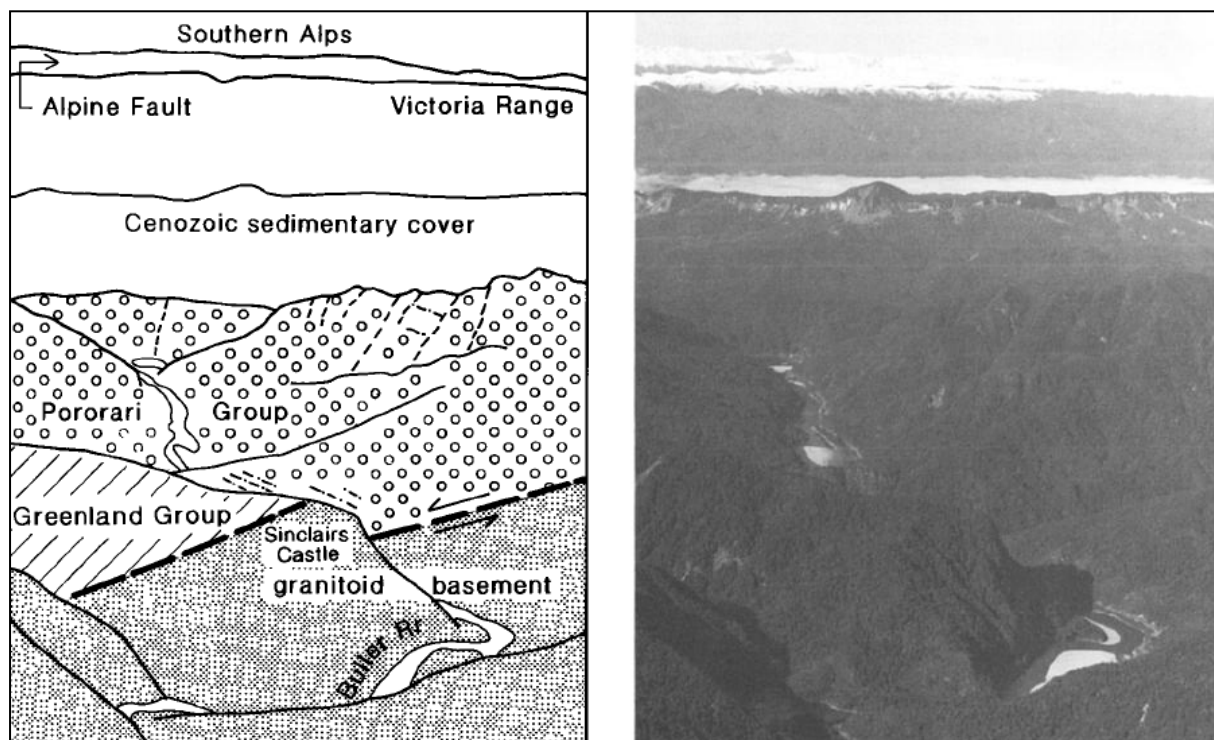
**Figure A5.7: View of Sinclair Castle (from Google Earth, image dated 1 April 2003)**

### Geology and geomorphology

Figure A5.8 shows the interpretation made by Tulloch and Kimbrough (1989) of the southwest tilting Pororari Group beds (dot/dashed lines) into the detached segment (heavy-dashed lines). High-angle normal faults within the Pororari Group are represented in Figure A5.8 by the comparatively lighter dashed lines. The oblique aerial view looking upstream shows the position of Sinclair Castle as a prominent outcrop of “granitoid basement” (terminology from Tulloch and Kimbrough, 1989), and the steep southeast-facing slopes forming the rear of the “Pre-1947 Landslide Area” are evident on the upstream side of Sinclair Castle in Figure A5.7.



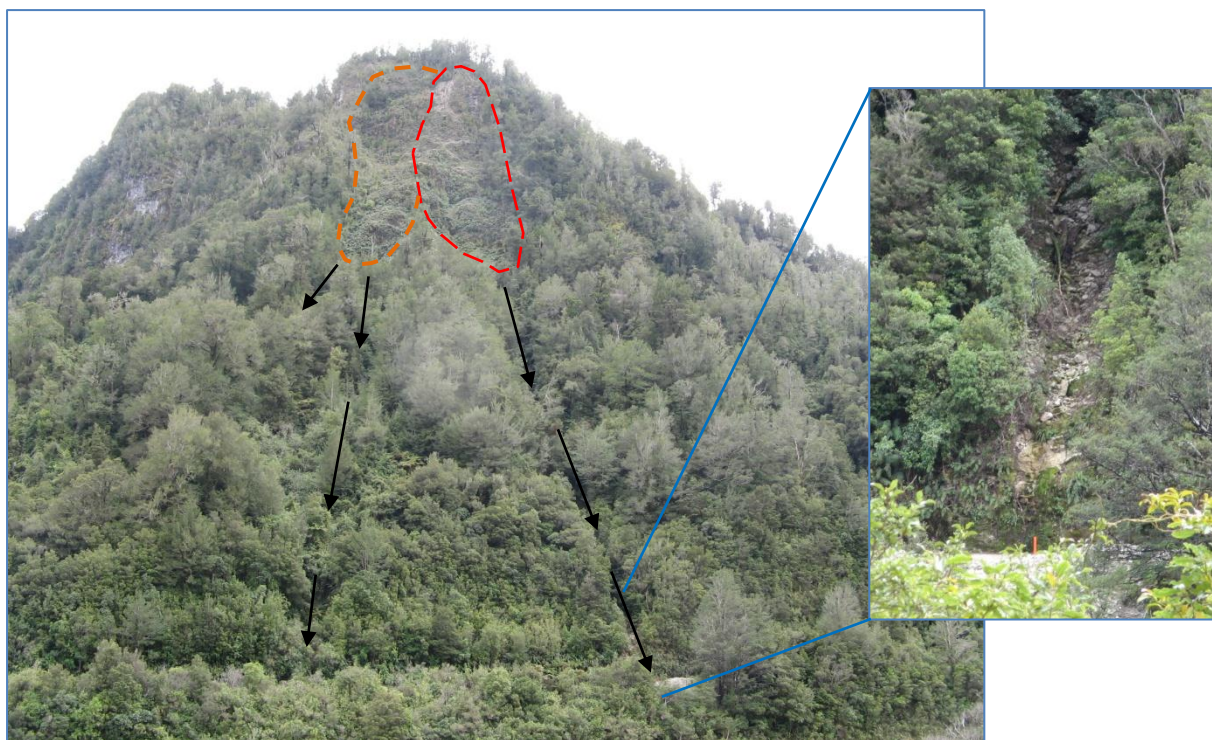
The granite in this location is assumed to have a high intact rock strength typical of other granite outcrops (unconfined compressive strength >50MPa). Joint spacing is variable, but typically of the order of 0.5 to 5.0m with persistence of metres to tens of metres, and there is no preferred orientation. In the vicinity of Sinclair Castle the intact rock is slightly to moderately weathered, with little penetration of chemical alteration away from the often iron-stained joints. The displaced bedrock within the “Pre-1947 Landslide Area” is slightly to moderately weathered, but still strong (UCS >50MPa) with steep slopes (>35°, refer Chapter 4, Figure 4.2). The rock behaves geotechnically in a similar manner to the granite exposed in the headscarp area, with gully-controlled features having formed in more closely fractured zones towards the eastern limit of the pre-1947 landslide.



**Figure A5.8: Aerial view from Tulloch and Kimbrough (1989) of Sinclair Castle looking towards the east up the Lower Buller Gorge. Refer text for description.**

### Slope Stability Assessment

Weathering processes, defect-controlled block release from the steep to very steep slopes, and the high annual rainfall have combined to trigger numerous block slides and debris avalanches since the construction of the rail in the early to mid-1940s. These are particularly associated with the eastern extent of the “Pre-1947 Landslide Area” failure shown in Figure A5.7, and involve block detachment from the upper source areas (Figure A5.8). The most recent failures occurred in 2011 and remedial works in December 2011 were aimed at temporary mitigation of the slope failure by scaling loose rocks, and by assessing the requirements for longer term stability. Deep-seated movement is not occurring, and instability is only indirectly related to the “Pre-1947 Landslide Area” (Figure A5.7).



**Figure A5.9: View of failure surfaces outlined by orange and red dashed lines (source areas) at Sinclair Castle and travel path (track zone) of material that intersected the rail corridor. The red dashed lines indicate a comparatively more recent event in 2011, including the inset location at rail level.**

The view in Figure A5.9 identifies the nature of present instability at Sinclair Castle. The source areas are very steep ( $>35^\circ$ ), and episodically release granite blocks up to  $5\text{m}^3$  in volume by near-surface weathering processes and joint-controlled dilation. These blocks then progress by a combination of falling, bouncing and/or sliding through the track zone mid-slope, with potential flows in gully areas closer to the railway (inset in Figure A5.9). The run-out zone extends naturally downslope to the rail corridor. Episodic release of rock debris by either earthquake shaking or rainstorm-induced regolith instability will continue to cause localised problems at the foot of the steep vegetated slopes, and there are no simple or inexpensive remedial options.

### Long-term Management

To address the continuing hazard to rail operations in the vicinity of Sinclair Castle around SNL 118.50km, it is necessary to document and analyse the past and present instability, and in particular to identify the run-out zones that are affecting safe movement of trains (including the examples shown on Figure A5.9 by the black arrows). As with similar landsliding problems on transportation routes (Turner and Schuster, 1996), the following options for remediation are available:

- Removal or stabilisation of loose granitic rocks within the source area, with associated safety considerations such as temporary closure and protection of the railway.

- 
- Construction of check structures within the track zone above the railway, particularly across gullies if these prove critical. The absence of a formed access to enable machinery to implement this option is recognised.
  - Avalanche-type chutes that route debris over the railway. Similar examples exist in North America (Turner and Schuster, 1996), and New Zealand (State Highway 73 in the Otira Gorge).
  - Construction of a shelter structure over a section of the railway line, if the severity and/or frequency of track closure warranted such measures (although this is an expensive option).
  - Acceptance of the risk to rail operations, and implementation of a series of protocols for regular inspection, culvert clearance and minor works to minimise closure time.
  - Any combination of these measures, commensurate with the perceived risk and economic consequences of rail closure (for example, after a large-magnitude earthquake).

Given the extent of rail traffic, and the consequences of closure, it is suggested that a combination of regular inspection, culvert clearance, minor works, and source area stabilisation are evaluated as the preferred short-term solution at SNL118.50km.

### **CASE STUDY 3: SNL121.00km – Cascade**

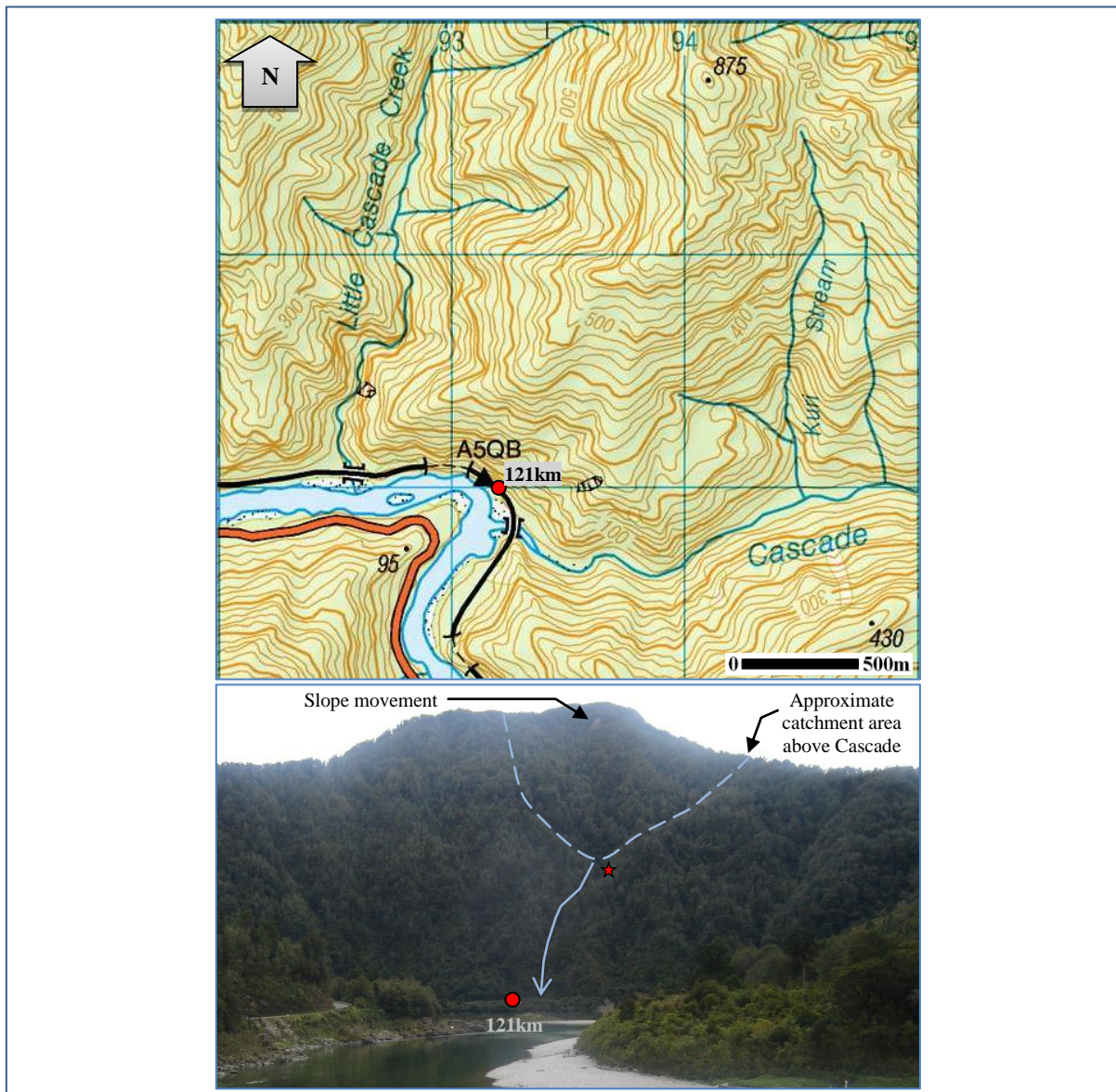
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The location referred to as Cascade in this section is shown in Figure A5.10, and it specifically includes the catchment area immediately above the rail at SNL121.0km that has sourced numerous blocky debris slides and flows since the rail became operational. This section of rail was first developed in the early 1900s to transport coal from Denniston Plateau, which was flumed down Cascade Creek. The original coal bins are still visible a short distance along the siding adjacent to Cascade Creek. In the 1940s the railway line was constructed through to Inangahua Junction, and linked through to the Reefton line from Stillwater Junction.

#### **Hazard description and remedial works**

In response to frequent (typically once every two years) disruption to rail operations, a diversion tunnel was constructed after flooding events in the 1980s to direct the flow of water from one gully to another. This effectively relieves the accumulation, and subsequent mobilisation, of rock debris within the catchment immediately above SNL121.00km. The indicative location of the diversion tunnel is shown by the star symbol in Figure A5.10. A manually constructed boulder dam was established to divert water into the tunnel. Annual inspections are carried out to ensure the tunnel entrance does not become blocked.





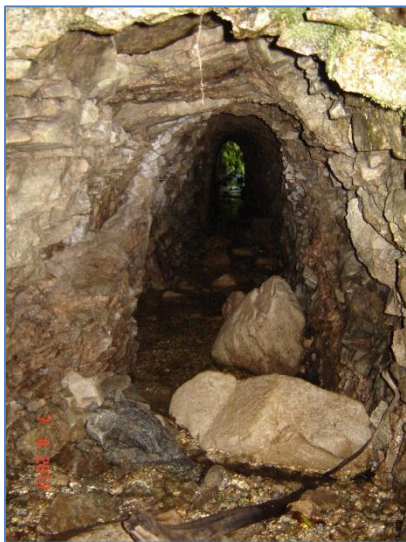
**Figure A5.10: Topographic contours in the vicinity of Cascade (top) and photograph taken from SH6 looking towards the catchment influencing slope stability at Cascade (bottom). Star indicates approximate location of the diversion tunnel.**

### Catchment Geology and Geomorphology

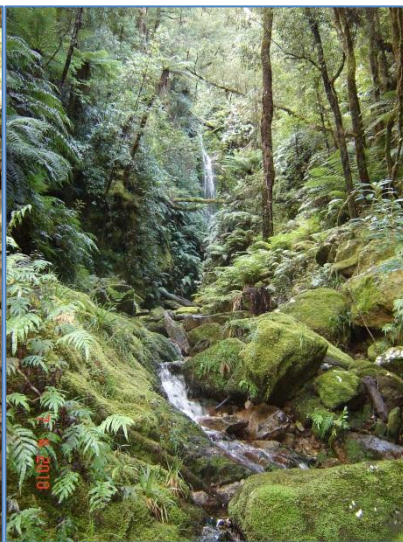
The steep slopes (typically  $\geq 30^\circ$ ) at SNL121.0km are formed on granites, and the tunnel portal (Figure A5.11) exposes jointed granite that can be described as: “*brownish grey, fine-grained granite; slightly weathered; strong and hard; joint-spacing 0.5 to 2m*”. The thickness of regolith development is variable, but typically less than 3m thick except where localised accumulation of weathered debris has resulted from shallow translational landsliding. Figures A5.11 and A5.12 illustrate the typical blocky debris that has accumulated from joint-controlled erosion of granite, and the stream morphology near the diversion tunnel. The finer-grained material is the result of *in situ* weathering of the granite, and subsequent colluvial accumulation on the steep slopes.

Figure A5.10 shows the steep nature of the bush-covered slopes above SNL121.0km, and the extent of the catchment that feeds the partly incised gully system on the lower slope. The size of the granite blocks that have been transported through the system is evident in Figures A5.11 and A5.12, and boulders exceeding 1m<sup>3</sup> were present upstream of the tunnel portal site. Areas of slope movement in the upper catchments are visible in Figure A5.10 (arrowed), and localised shallow failures in near-surface weathered granite were evident in the vicinity of the diversion tunnel itself. This is part of the normal weathering cycle in granite bedrock on steep slopes with high annual rainfall, and episodic extreme storm events involving regolith saturation and translational sliding at the bedrock interface.

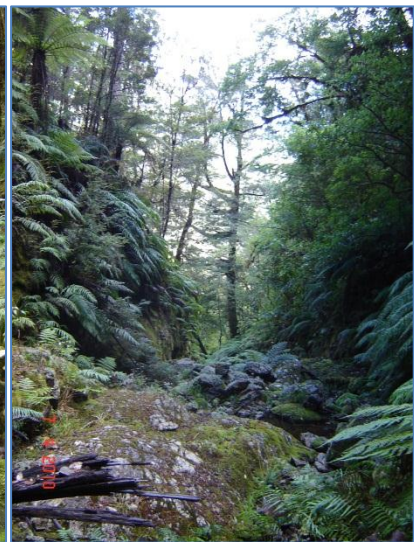
The decision to undertake stream diversion in the 1980s by constructing an approximately 35m long tunnel was carried out to improve the control of overland water flow. One incident was recalled post-1990 (per comms, Gary Hegan, September 2010) that involved the diversion tunnel becoming blocked and associated overtopping of the manually constructed boulder dam. This incident impacted rail operations and tunnel clearance was carried out as soon as practicable.



**Figure A5.11: Diversion tunnel entrance (~1.5m in height)**



**Figure A5.12: View looking upstream from diversion tunnel**



**Figure A5.13: View looking downstream and edge of rock boulder dam**

### **Hazard and Consequence Evaluation**

The hazard at the Cascade site (SNL121.0km) is the mobilisation of blocky debris by slide-flow processes within a steep ( $\geq 35^\circ$ ) catchment, and the transfer of this material through the gully system during extreme storm conditions causing culvert blockage and track damage affecting rail operations. Anecdotal data from KiwiRail suggests a frequency of about once every two years for such events, and this was the basis for diversion of the stream to a more easily controlled outlet. Debris volumes of tens to hundreds of cubic metres may be involved in individual storm events, and the block size suggests potentially significant impacts to rail operations.



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An annualised probability of 0.5 justified the measures undertaken above Cascade, and anecdotal information from KiwiRail indicates that the frequency of track closure at this site has been reduced to less than once every 20 years ( $p < 0.05$ ) by the measures implemented. Were the diversion tunnel portal to block, the previous rail management problems would recur, and there is no access to the tunnel site for earthmoving machinery in the event of an emergency. Given the size of block evident in the stream bed and at the tunnel portal (Figures A5.11 and A5.12), the consequences of blockage are such that improved future management practices will likely still be required.

This case study highlights the difficulties in managing the railway corridor in such steep rugged terrain, where extreme runoff can result in debris mobilisation within the catchment both by shallow landsliding on riparian slopes and by reactivation of debris within channels or banks. The consequences of rapid channelised debris movement on slopes  $\geq 35^\circ$  justify measures such as those carried out to minimise the impacts on rail operations and the safety of KiwiRail staff and contractors.

### **Future Site Management**

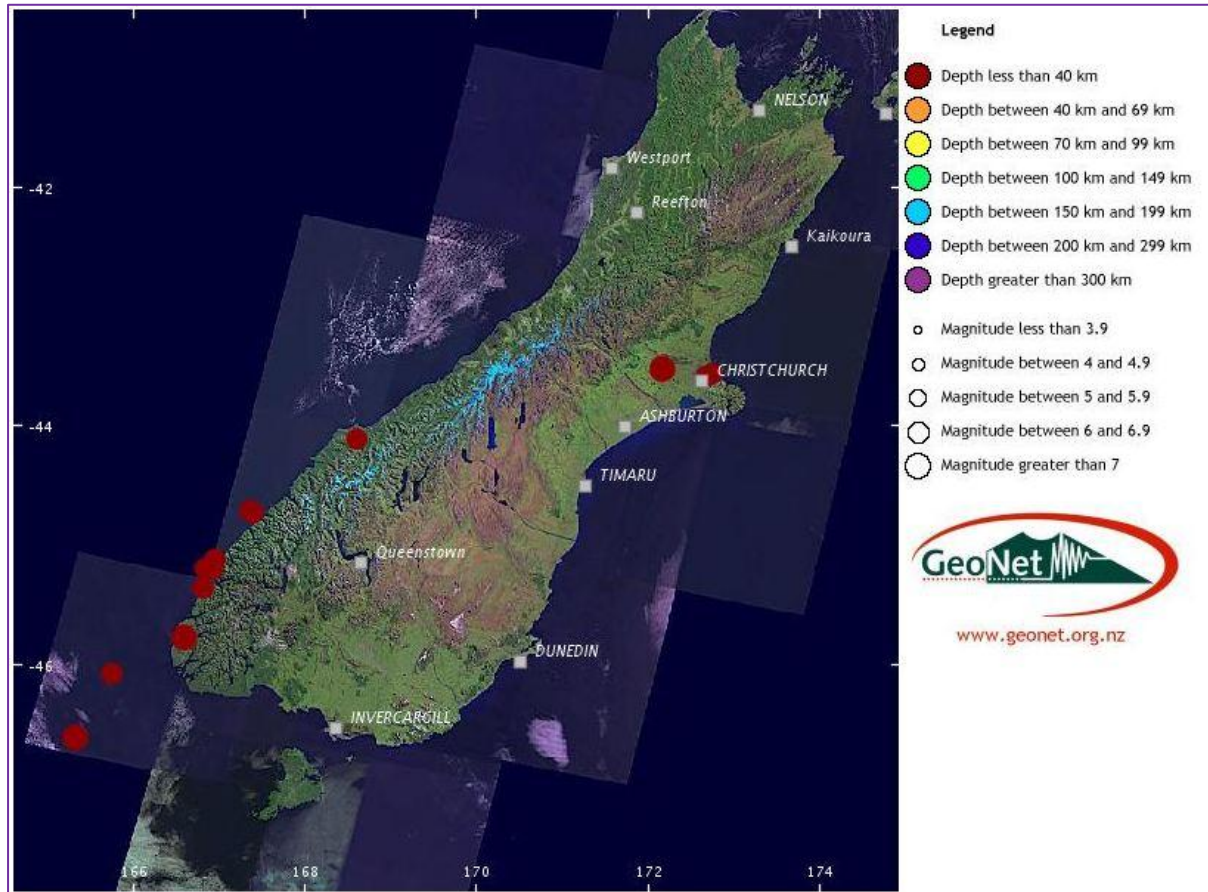
The key aspect of the present diversion tunnel construction is that rapid slides and flows of blocky debris within the steep ( $>35^\circ$ ) stream channel above SNL121.0km have been effectively prevented, and therefore the immediate risk to rail operations has been minimised. The remedial works do not preclude future failures within other nearby gullies, or additional slope failures affecting track operations. In terms of long-term Landslide Risk Management at Cascade, the following points should be considered:

- The absence of machine access to the diversion tunnel precludes remedial measures, other than manual, to maintain the site, or to repair, as required, the non-engineered boulder dam that is currently controlling water flow (Figure A5.13).
- Annual maintenance inspections are inadequate in the event of an extreme (for example 1 in 10-year return period) storm event, and protocols are advised for such situations.
- Redesign and armouring of the existing boulder dam, with provision for storm overflow, should be undertaken as a priority, together with a detention basin upstream of the tunnel inlet.
- Large boulders removed from the tunnel inlet (Figure A5.11) could be used for armouring as required, the granite blocks being hard, strong and durable materials suitable as armour.

In summary, the importance of maintaining the successful diversion is emphasised due to the susceptibility for blockages as a result of either extreme storm events or large magnitude earthquakes. Joint-controlled block failures around the diversion tunnel may occur under severe shaking conditions, and the consequences of renewed debris movement down the former stream channel would disrupt rail operations as it has in the past.

## Appendix 5.2

*Distribution of magnitude 6 and greater earthquakes in the South Island during the period January 1997 to August 2011*



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# Appendix 6.1

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## *Background information regarding the LiDAR survey conducted at Whitecliffs*

### Data Supply Metadata s1

<b>Project</b>	Whitecliffs Buller Gorge LiDAR	10.039
<b>Client</b>	Davis Ogilvie and Partners Ltd	
<b>Client Contact</b>	Lloyd McGarvey	

<b>Summary of Data</b>	<p>NZ Aerial Mapping (NZAM) collected LiDAR sensor data over the Whitecliffs area of interest totalling approx 0.5 sq km. The data was processed into various digital map data products. The products include in this data supply are:</p> <ul style="list-style-type: none"><li>• Project extent data</li><li>• DTM point cloud</li><li>• Above ground point cloud</li><li>• Contours</li><li>• Orthophotos</li><li>• Unclassified point cloud</li></ul> <p>Please refer to the report section <i>Product Generation and Data Supply</i> for details on these products.</p>
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<b>Data Acquisition</b>	<p>The project area over a section of the Whitecliffs is that shown in the ESRI shape file “whitecliffs_nztm” that accompanies the dataset. The total project is approximately 0.5 sq km in area. A map showing this area of interest is included in Appendix A.</p> <p>LiDAR and digital imagery was collected on 29 March 2010, using NZ Aerial Mapping’s Optech ALTM 3100EA LiDAR system and Rollei AIC medium format digital camera.</p> <p>The data was collected flying 1,200 metres above the ground, and using a field of view of 40 degrees. The system PRF was set at 70kHz. A geodetic reference mark that NZAM established at Westport airfield was used for the collection of GPS receiver station data during the aerial data acquisition.</p> <p>Independent of the aerial survey work, Davis Ogilvie field surveyed check sites, to be later used to verify the accuracy of the processed ground dataset. The field survey work was undertaken on 13 April 2010. This work was tied into the LINZ geodetic reference marks ADAQ, ASQW, OIS IX SO11778, OIT VII SO 11778 and AOAR.</p>
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<p><b>Data Processing</b></p>	<p>The LiDAR sensor positioning and orientation (POS) was determined using the collected GPS/IMU datasets and Applanix POSPac software. This work was all undertaken in NZGD2000 coordinate system using the data collected at the geodetic reference mark for the DGPS processing.</p> <p>The POS data was combined with the LiDAR range files and used to generate LiDAR point clouds in New Zealand Transverse Mercator (NZTM) map projection but NZGD2000 ellipsoidal heights. This process was undertaken using Optech DASHMap LiDAR processing software. The data was checked for completeness of coverage. The relative fit of data in the overlap between strips was also checked. The point cloud data was then classified into ground, first and, intermediate returns using automated routines tailored to the project landcover and terrain. The subsequent steps were undertaken using TerraSolid LiDAR processing software modules TerraScan, TerraPhoto and TerraModeler.</p> <p>The Rollei camera images were developed into 8 bit per channel uncompressed TIFF format images. The LiDAR POS data was transformed for use with the camera, and this data was used with the automated classified ground LiDAR point cloud data to produce orthophotos with a ground sample distance of 0.15m.</p> <p>The data was converted from NZGD2000 ellipsoidal heights into orthometric heights using the LINZ NZGeoid09 separation and offset model.</p> <p>Comprehensive manual editing of the LiDAR point cloud data was undertaken to increase the quality of the automatically classified ground point dataset. This editing involved visually checking over the data and changing the classification of points into and out of the ground point dataset. Attention was particularly focused on hydrologic features and areas of vegetation. The Rollei orthophotos were used as a backdrop when undertaking the manual editing. As part of the manual edit process LiDAR returns from water bodies were removed from the ground point dataset and placed in their own dataset. Much of cliff face by the railway line is undercut. To enable contours to be produced from the point cloud the LiDAR returns from the undercut areas were moved into their own data class – Obscured.</p> <p>The height accuracy of the ground classified LiDAR point was checked using open land-cover survey check site dataset that Davis Ogilvie surveyed at one location. This was done by calculating height difference statistics between a TIN of the LIDAR ground points and the checkpoints. The standard deviation statistic is +/-0.03m.</p> <p>The positional accuracy of the LiDAR data has been checked by overlaying Davis Ogilvie surveyed data over the LIDAR data displayed coded by intensity. The data was found to fit well in position.</p>
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<b>Product Generation &amp; Data Supply</b>	<p>The supplied products are all in terms of New Zealand Transverse Mercator (NZTM) map projection, and Lyttelton Vertical Datum 1937. As the area of interest is small each of the products was output into a single file. The point cloud datasets are all in ASCII XYZ (mE mN O) file format.</p> <p>The folder <i>DTM</i> contains the LiDAR point cloud points that have been classified as ground returns as well as supplementary points added to the dataset (<i>Whitecliffs_DTM.xyz</i>). The point cloud points identified in the undercut sections of the cliff face are in the file <i>Whitecliffs_Obsc.xyz</i></p> <p>The folder <i>Above Ground</i> contains the LiDAR point cloud points that have been identified as having elevations greater than the points in the DTM dataset.</p> <p>The folder <i>Unclassified</i> contains all the LiDAR point cloud points that were collected by the LiDAR sensor.</p> <p>The folder <i>Contours</i> contains 1m contour interval contours. The contours were interpolated from a TIN created using the <i>Whitecliffs_DTM</i> LiDAR point cloud dataset. The 5m interval contours have the TYPE attribute set to INDEX. The rest of the contours are TYPE – Intermediate. This data is provided in both ESRI shape and AutoCAD DXF file formats.</p> <p>The folder <i>Orthophotos</i> contains the 0.15m GSD orthophoto produced using the Rollei AIC camera imagery. This orthophoto is in TIFF/ESRI TFW file format. The Rollei AIC camera is a semi-metric camera and so the orthophotos contain mosaic mismatches. They are provided as a general point of reference only and are not suitable for accuracy measurement.</p> <p>The folder <i>Cadastral</i> contains property boundary lines extracted from TerraView (date of data November 2009). This data is provided in both ESRI shape and AutoCAD DXF file formats.</p> <p>If you have requirements for the data in other file formats, map projections please contact NZAM. Our contact details are provided below.</p>
<b>Exceptions</b>	<p>No exceptions with the dataset have been noted.</p>



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<b>Supplier</b>	NZ Aerial Mapping Ltd
<b>Address</b>	208 Warren Street PO Box 6 Hastings 4158 New Zealand
<b>Phone</b>	64-6-873 7550
<b>Supplier Contact</b>	Dave Froggatt (dave.froggatt@nzam.com)
<b>Date of Metadata Creation</b>	21 April 2010
<b>Author</b>	Tim Farrier